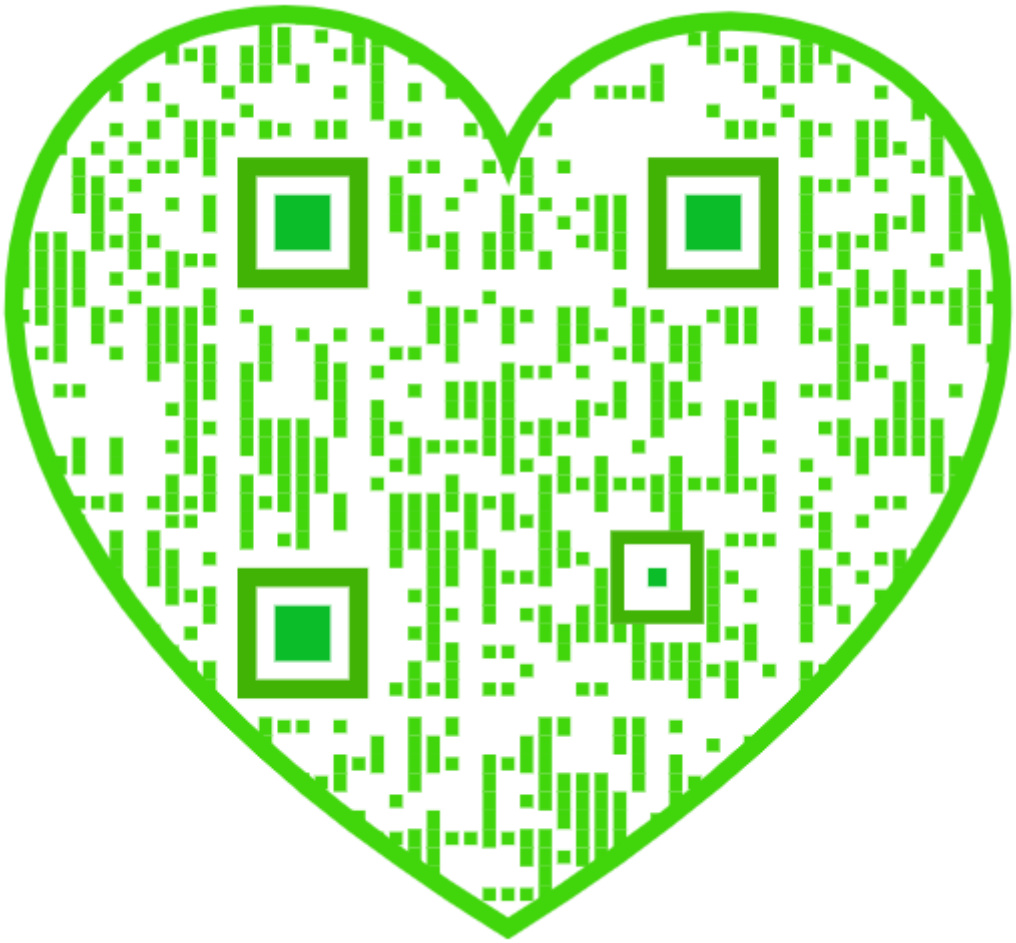

J. S. Falk

SEWAGE DISPOSAL

BY

LEONARD P. KINNICUTT

Late Director Department of Chemistry, and Professor of Sanitary Chemistry in the Worcester Polytechnic Institute

C.-E. A. WINSLOW

Professor of Public Health in the Yale School of Medicine; and Curator of Public Health in the American Museum of Natural History, New York

AND

R. WINTHROP PRATT

Consulting Engineer, Member American Society of Civil Engineers

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PREFACE TO THE FIRST EDITION

When this book was begun, neither H. P. Raikes' "Design, Construction and Maintenance of Sewage Disposal Works" nor H. T. Calvert's translation of Dunbar's "Principles of Sewage Treatment" had been published. Even after these excellent books appeared, however, it still seemed to us that there remained room for a general survey of the problem from the various viewpoints of the chemist, the sanitary biologist and the engineer, and with particular reference to the conditions of American practice. It has been our aim to discuss somewhat fully the fundamental principles of chemistry and bacteriology which are involved and yet to include also the more important aspects of the engineering works designed to carry them into operation. It is hoped that the book may be useful to the student of sanitary engineering who aims to fit himself for the construction of sewage disposal works, to the engineer who after working in other lines is drawn into this growing field, and to the chemist, the bacteriologist and the public health official concerned in the operation of disposal works after they are built.

The thanks of the authors are due to a long list of engineers and others who have furnished unpublished data for various sections of the book. In particular, grateful mention must be made of Mr. J. D. Watson, of Birmingham, England; Mr. G. J. Fowler, of Manchester, England; Mr. W. J. Dibdin, of London, England; Mr. H. P. Eddy, Mr. Leonard Metcalf and Mr. F. A. Barbour, of Boston; Mr. G. E. Bolling, of Brockton, Mass.; Mr. W. M. Brown, of the Metropolitan Sewerage Commission of Massachusetts; Mr. A. L. Fales, supervising chemist Worcester Sewage Works; Mr. J. W. Bugbee, supervising chemist Providence Sewage Works; Mr. Charles Saville, of the Massachusetts State Board of Health, Mr. F. W. Jones, Worcester Polytechnic Institute, and Mr. Paul Hansen, of the Ohio State Board of Health.

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PREFACE TO THE SECOND EDITION

Progress in the art of sewage treatment has been rapid during the past few years. The activated sludge process has been developed to a point of practical importance. The two-story tank for the removal of suspended solids has come into general use and its advantages and disadvantages are more and more clearly understood. Improvements have been made in many lines, in the fine screening of sewage, in the disposal of sewage sludge, and in the recovery from sewage and sludge of its valuable constituents, nitrogen and grease. These advances have made necessary a radical revision of the present work.

The occasion has been taken to rewrite the last edition completely so as to bring in new data and recent viewpoints in connection with all the topics treated. In particular, the chapters on Screening, and on Sludge Disposal and on Chemical Precipitation (with a discussion of Boston and New Haven studies of the Miles Acid Process) have been entirely reconstructed and much enlarged. New chapters have been introduced, treating of the newer processes of two-story tank treatment and purification by means of activated sludge (including the results of Cleveland, Chicago and Milwaukee experience); and two other new chapters have been added which discuss, respectively, the broad principles underlying the selection, design and operation of a sewage treatment plant and the disposal of excretal wastes from houses and institutions which are not connected with sewerage systems.

In the present revision we have preserved the same general plan of approach adopted in earlier printings. We have laid emphasis throughout upon broad general principles and have attempted to trace the historic evolution of these principles in the work of the early investigators who contributed to each important branch of the subject; for such a broad historical background forms the surest basis for real comprehension of the present. On the other hand we have freely used detailed descrip-

tions of the engineering details of typical plants and extensive analytical tables, both chemical and bacteriological, as illustrative material to reënforce the general principles presented in the text.

While the demands of the war have caused in many cases postponement of sewerage and sewage disposal projects which would otherwise have been undertaken, the growth of munition towns and shipyards has, on the other hand, created new sewage treatment problems of a most acute kind. After the war, both in this country and in the stricken countries of Europe and Asia, which we must help to reconstruct, safe and economical disposal of sewage is sure to be a pressing necessity; and in the new work which will be demanded many engineers and contractors, physicians and public health workers, not specifically trained in the special field of sanitary engineering, will be concerned. It is in the hope that this presentation of the principles of sewage treatment may be of assistance to such workers that the authors have prepared it at this time.

C.-E. A. WINSLOW.
R. W. PRATT.

September, 1918.

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SEWAGE DISPOSAL

INTRODUCTION

THE SANITARY DEMAND FOR SEWERAGE AND SEWAGE DISPOSAL

The Wastes of Human Life. The human body is a biological machine which requires food for fuel and which produces in its operation a considerable amount of waste material. Like the ash from a furnace, this waste consists partly of mineral matter and partly of incompletely oxidized fuel. The excretions of the kidneys, representing the end products of cell metabolism, still contain a large amount of organic matter in the form of urea; the discharges from the alimentary tract consist largely of undigested, or partially digested, foods which have not been absorbed by the body. All these substances undergo further change after they are excreted from the body, breaking down into simpler or more stable compounds, and during this change compounds are formed which are characterized by the penetrating noxious odors of putrefaction.

Besides the organic waste materials from digestion and excretion, the feces contain a host of microscopic living organisms, and this fauna and flora may be of even greater practical importance than the lifeless substratum upon which the organisms subsist. The surfaces of the human body, within and without, are parasitized by micro-organisms, which find their most favorable conditions for multiplication in the digestive tract. MacNeal, Latzer and Kerr (1909) report an average of 33 million million bacteria per day in the feces of normal adult men;* and Protozoa, though not so ubiquitous, are by no means rare. Most of

* Complete references to all literature cited will be found in the bibliography at the end. References in the text include the name of the authority (the initials in the case of the British commissions) and the date of publication, with a distinguishing letter in case more than one volume appeared in a single year. This serves simply to identify the article or book, the full title of which is given in the bibliography.

these microscopic parasites are harmless, putrefactive forms; but in the excreta of a patient suffering from typhoid fever, or dysentery, or cholera, or hookworm, or any other intestinal malady, the specific germ of the disease may at any time be present.

The discharges from the alimentary canal and the kidneys do not of course exhaust the catalogue of human wastes. The washings of the outer skin, and the wash water from cooking and house-cleaning, are included in the wastes from the household. In more closely settled communities, street washings and the wastes from industrial establishments are also added. With the exception of factory wastes and street washings, which are of various composition and require special treatment, the excretions of the body may, however, be taken as a general type. The important constituents in every case are the intermediate products of organic decomposition, plus the living micro-organisms, which may at any time include specific pathogenic forms.

Primitive Methods of Dealing with Excreta. Under primitive conditions excretal matter ultimately finds its way to the water or the earth. Direct discharge into watercourses is eminently satisfactory to the persons immediately concerned, if the flow be sufficient to prevent local accumulations. The effect of this procedure upon individuals and communities on the watercourse below may, however, be serious. This is an aspect of the larger problem of sewage disposal and stream pollution which will be discussed more fully further on; as far as the polluting individual goes, direct discharge into water is an efficient method of disposal. The earth, too, is able to assimilate decomposing organic matter with success, as demonstrated yearly by the manuring of the fields. Mixed with a sufficient quantity of earth, and with reasonably free access of air, excreta are quickly disintegrated and oxidized to stable and innocuous forms.

The difficulty with the method of earth disposal lies in its application. Its success demands prompt and complete mixture with clean dry earth; and this is rarely attained. The conditions which actually exist are various. The most primitive houses are provided with privy vaults for the excreta, but discharge sink and other wastes directly on the surface of the ground, with or without the medium of a drain. Combinations of privy vaults for excreta with cesspools for sink drain-

age lead up to a better class of houses provided with cesspool connections for all wastes.

The privy vault is certainly the most objectionable of these contrivances. It stores up quantities of human excreta in a slowly decomposing condition. It is generally loosely built, and the material which it contains is more or less freely exposed to the air and to the distributing agency of insects and higher animals. It is subject to overflow and surface discharge at times of heavy rain; and the material which it contains must generally

Fig. 1. Removal of Night-soil, Tokyo.

be removed and otherwise disposed of at intervals, the process of handling causing fresh nuisance and menace to health. A well-constructed cesspool is free from many of these objections. The material in it is, or should be, closed in, so that air-borne odors and the access of insects are prevented. If the surrounding soil be of a suitable sand, the liquid contents of the cesspool may so filter through it as to be efficiently purified. Such a cesspool sometimes operates for years without filling up and without causing pollution of water or earth. Where, however, a rocky or clayey soil is traversed by fissures, a leaching

cesspool may constitute a serious menace to well waters, even at considerable distances. A tight-walled cesspool, on the other hand, resembles a privy vault in the danger that it may overflow and in the certainty that it must be emptied and the material in it removed and carted away.

Throughout the Far East, where animal manure is scarce, and where the crowded populations depend for their existence upon intensive agriculture, the dry system is almost universally relied upon and the night soil is carried off to be used as fertilizer on the fields. In Japan the typical closet seat is a slit in the floor, often provided with a straight back at one end to lean against; and this slit discharges into a vault below. As a rule the vault extends out beyond the wall of the house and the contents are removed from outside by means of a dipper on the end of a long pole. Fig. 1 shows how the material thus dipped out is carried away from the house in pails.

Similar systems prevail even in certain European countries and notably in Russia. Scarcely a dozen Russian cities have complete systems of sanitary sewerage. In Petrograd though there is an extensive system of sewers they are chiefly for carrying storm water and do not receive any large proportion of fecal wastes. The latter are, for the most part, collected in cesspools which are pumped out at night (at intervals varying from 2 weeks to several months) into metal tank carts (see Fig. 2) which convey the sewage liquids to a central disposal station where they are screened through Riensch-Wurl screens, the effluent being discharged through a 10-inch sewer at a point 7 kilometers down the harbor.

The best available methods for disposing of the sewage materials from an individual household will be discussed in Chap. XVI; but all procedures which involve the retention of excretal matters in or near dwellings must be considered as temporary makeshifts. They may operate well under rigid supervision, but in the long run are likely to give rise to nuisance and to endanger health.

The Danger in Accumulations of Excretal Materials. Accumulations of excretal matter are not objectionable, simply on account of the unpleasant odors due to their decomposition. The nuisance from an individual cesspool or sink-drain is seldom sufficient to be obnoxious, except in the immediate vicinity.

The real danger lies in the disease germs which may be present. Typhoid fever and dysentery are unfortunately still common diseases; and in the discharge of persons suffering from these, or other, intestinal disorders the specific parasites may be present in great numbers. Even with patients patently suffering from

Fig. 2. Tank Cart for Removal of Cesspool Contents, Petrograd.

such maladies it is rare that proper precautions are taken for disinfection. Unrecognized cases of a light nature may spread the most virulent germs; and occasionally " typhoid carriers " are found, infected with pathogenic organisms, though not themselves suffering from the disease. Wherever excreta are present the germs of intestinal disease are potentially present also.

If excretal matter carrying disease germs is kept exposed in privy vaults, or is discharged from overflowing cesspools, the transfer of the infection to susceptible human beings is one of the most probable of events. Flies, which impartially affect privies and larders, offer perhaps the most convenient mechanism for transferring fecal matter to the mouth. Rats and chickens and other animals play their part. Children may easily acquire more direct infection. The carelessness which is encouraged by the uncleanly environment of the privy vault

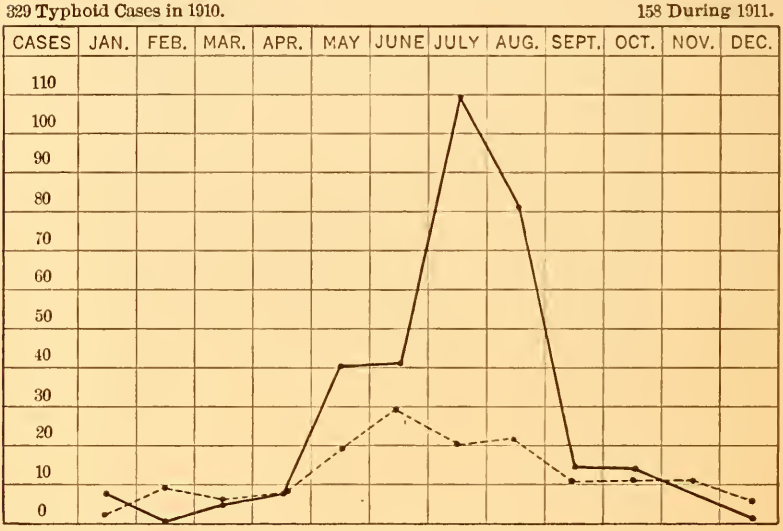

FIG. 3. Typhoid Fever at Jacksonville, Fla. Heavy line before, dotted line after, closets were made fly-proof.

increases the danger to all who use it; and, finally, the emptying of privy or cesspool and the transportation of the contained material to its point of final disposal spreads infection in a hundred ways.

The statistics of typhoid fever in the Spanish-American war offered striking proof, if any were needed, of the relation between that disease and the care of excreta. The chief cause of the scandalous prevalence of typhoid (which affected one-fifth of all the troops in the national encampments of the United States) was camp pollution. The official report of Surgeons Reed, Vaughan and Shakespeare (1904) concludes, " It may be

stated in a general way that the number of cases of typhoid fever in the different camps varied with the methods of disposing of the excretions." In the Seventh Army Corps, for example, the First Division had a water-carriage system of disposal and developed 173 cases of typhoid fever per regiment, on the average. In the Third Division regulation pits were used, in which the excreta were supposed to be promptly covered with earth. This method is fairly satisfactory, though inferior to water carriage. There were 185 cases per regiment in this Division. The Second Division used the tub system of disposal, by which "infected fecal matter was scattered all through the camp," and had an average of 299 cases per regiment.

The experience of the city of Jacksonville, Fla., is another interesting illustration of the evil effects of improper disposal of excretal wastes. This city was for years heavily infected with typhoid fever, largely as a result of the proximity of the army camps in 1898. For the years 1908, 1909 and 1910, the typhoid death rates of the city were 82, 75 and 106 respectively. The Health Officer, Dr. C. E. Terry, believed that defective privies were the chief factor in spreading the disease, and in the late summer of 1910 a law was passed requiring that all dry closets within the city be rendered fly-proof. By March 1911 about 75 per cent, and by Jan. 1912 practically all, of the privies had been brought into conformity with this law, and the typhoid rates for the years 1911 and 1912 dropped to 63 and 26, respectively.

The Water-carriage System. Where a water-supply is not available it is obviously impossible to secure the immediate removal of excreta, and the only thing to do is to minimize as far as possible the dangers which arise. For this purpose various methods of dry disposal have been devised, dating back at least to the excellent sanitary regulations in the twenty-third chapter of Deuteronomy; and some of these methods may be used with considerable success as in the case of Jacksonville just cited.

The ideal method of removing excreta is, however, the water-carriage system, in which the excreta are at once washed away into a system of closed pipes and removed promptly and completely from the vicinity of the dwelling.

The recognition of this fact is comparatively recent. The

Cloaca Maxima and the other so-called sewers of antiquity were rather drains than sewers, and their function was to lower the ground-water level and to carry surface water rather than to remove excreta. Until 1815 the discharge of any waste but kitchen slops into the drains of London was prohibited by law, and the same regulation persisted in Paris up to 1880. Sewerage and sewage disposal proper really date from the epoch-making report of the health of towns commission of Great Britain in 1844, which revealed the accumulation of such an astonishing amount of decomposing organic matter and filth that it aroused British sanitarians to a strong movement for the amelioration of these conditions. Public and private cleanliness was taught and practiced as never before. The midden system and the pail system rapidly gave way to the water-carriage system. Whereas in 1815 the sewers of London were simply drains to carry off the storm water, in 1847, only three years after the report of the health of towns commission, it was made obligatory to discharge all sewage into those drains.

In other countries the example set in England was more or less promptly followed. In the United States numerous drainage systems existed, — one in Boston, for example, dating from the seventeenth century; but the first comprehensive sewerage project was designed by E. S. Chesbrough for the city of Chicago in 1855. On the continent of Europe a sewer system was constructed at Hamburg after the great fire of 1842, by Lindley, an English engineer. Berlin began her sewerage in 1860 and other German systems quickly followed.

Efficient Sewerage Systems and the Death Rate. The new method of dealing with excreta quickly justified itself by its results. A marked decline in the death rate, and particularly in the typhoid rate, has followed the introduction of sewerage systems. In many cases simultaneous improvements in water supply complicate the results; but in a number of instances it seems clear that the removal of excreta was the main force at work. Thus Pettenkofer (1874) shows that at Munich the typhoid death rate was 242 per 100,000 between 1852 and 1859. Improvements in privy vaults and the construction of sewers began between 1856 and 1859. From 1860 to 1867 the typhoid death rate fell to 166. New water supplies and other reforms have since reduced the death rate to a very much lower point;

but this first diminution of one-third was primarily the result of sewerage.

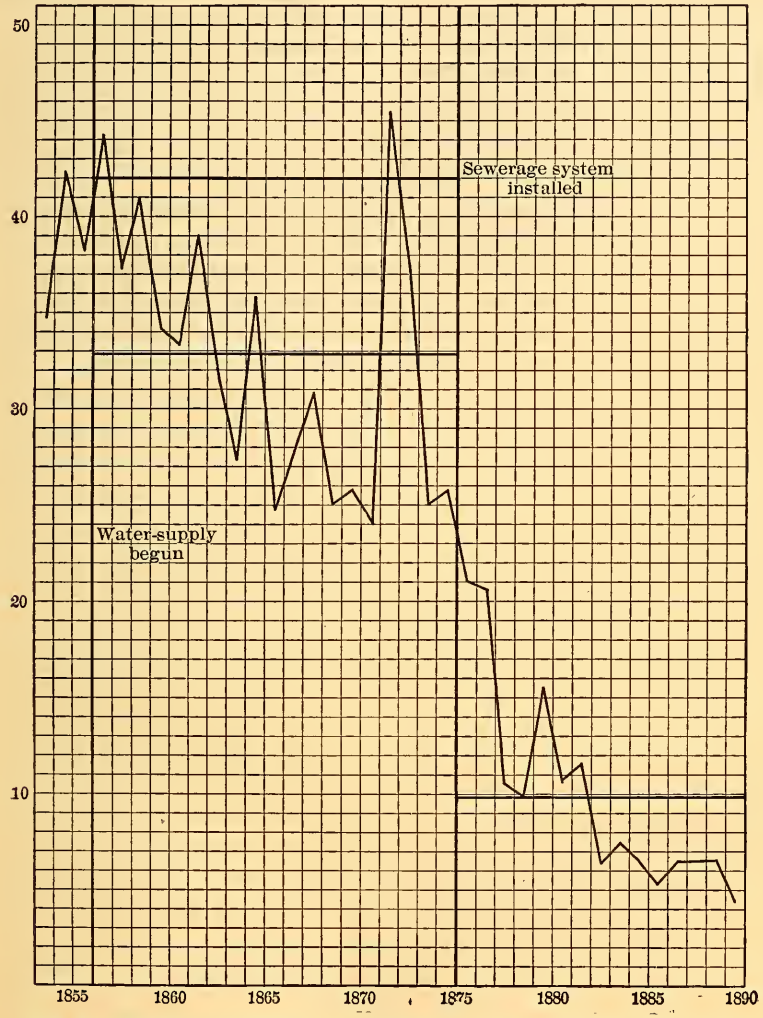

FIG. 4. Effect of Water Supply and Sewerage on the Typhoid Death Rate of Berlin (Blasius, 1894).

In Berlin, Weyl (1893) records similar phenomena. The introduction of a public water supply in 1856 produced a rapid decrease in typhoid fever; but the opening of the first consider-

able system of sewers in 1876 caused the curve of typhoid to take a much steeper fall than its previous course would have indicated. The curve for the ratio of typhoid deaths to total deaths in Berlin, during this period, is plotted in Fig. 4.

The Sanitary Significance of Drainage. There is another aspect of the sewerage problem which deserves some consideration — the sanitary importance of drainage, pure and simple, as distinct from sewerage. A sewerage system not only carries away excreta; it furnishes also, if desired, an opportunity for the removal of surface water; and it may be so constructed, or so supplemented by subdrains, as to effect a permanent lowering of the ground-water level. The removal of accumulations of surface water is obviously of much sanitary importance, since stagnant water offers an opportunity for the *Anopheles* mosquito to breed and thus promotes the spread of malaria. The lowering of subsoil water is also a sanitary desideratum under certain conditions. From the classic investigations of Buchanan in England (Buchanan, 1867) it appeared that where sewerage systems led to an appreciable drying of the soil a marked diminution of tuberculosis followed, whereas in towns where no such drying took place the tuberculosis death rate was stationary. In this case the direct effect of a drier air on vital resistance was perhaps the efficient cause.

The New Problem of Sewage Disposal. Water carriage is clearly the ideal method of sewerage for the individual householder. It removes excreta and all other liquid wastes promptly and completely from the region of habitation; it prevents contamination of water and earth; and it offers an opportunity for the drying of surface soil and subsoil. The problem of ultimate disposal is, however, merely shifted from the individual to the community. The unsanitary condition surrounding the dwelling is relieved, but at some point on the outskirts of the city the concentrated filth from the entire population must be delivered and must be taken care of by municipal authorities, and the problem is rendered all the more difficult by the large amount of water which carries this filth through the sewers to the point of discharge. The proper disposal of the combined waste of the community so that it shall not cause offensive or dangerous conditions, is the problem of sewage disposal. The magnitude of this problem may be realized by the simple statement that

five hundred million gallons of sewage a day containing 1600 tons of dry solid matter, are poured into New York Harbor from the sewers of the city. If this vast volume of liquid were collected in one place it would fill the East River under Brooklyn Bridge for a distance of one-fifth of a mile. How far the sewage problem still remains from being solved is indicated by the estimates of George M. Wisner of Chicago, that out of a total of 91,600,000 persons in the United States in 1910 only 34,700,000 or 38 per cent lived in communities provided with any sewerage systems at all, and only 3,900,000 or 4 per cent lived in communities provided with sewage treatment works, the remaining 30,800,000 discharging their sewage untreated into the nearest body of water. Metcalf and Eddy estimate that in 1915 the population disharging raw sewage into the ocean or into inland streams or lakes had risen to 34,900,000, while 6,900,000 persons contributed sewage to systems provided with treatment works.

Objects and Limitations of Sewage Treatment. It is possible by methods which we shall describe to purify any sewage to any desired degree. The suspended solids may be completely removed and the organic matter completely nitrified so as to produce a clear and sparkling effluent, resembling a spring water; and the bacterial purification may be carried so far as to make this effluent as pure as many public water supplies.

It should be borne in mind however that anything approaching such complete purification is very rarely necessary or economical. Health authorities have at times insisted on a far higher degree of purification of sewage than is justifiable on sound grounds of public policy. Municipal appropriations are limited and there is no reasonable excuse for spending, on ultra-refinements, money which can be used along other lines of public health endeavor to yield definite and practical results in the saving of human lives. For this reason Prof. George C. Whipple has wisely emphasized the desirability of using the term "Sewage Treatment" rather than Sewage Purification in describing the art with which we are here concerned.

Sewage treatment should in all cases be carried far enough to avoid local nuisance and the production of conditions offensive to sight and smell. In some cases this end may be attained by simple screening combined with disposal by dilution through properly located outlets. In other instances removal of finer

suspended solids by tank treatment may be necessary, in still others treatment in oxidizing beds, the degree of purification depending upon the digestive capacity of adjacent bodies of water. In certain cases where only very small streams are available a degree of purity such as is attained by intermittent filtration through fine grain filters may prove essential. The particular combination of processes which will prove best in a given case can only be determined by careful study of local conditions. There is no "system" of sewage disposal any more than there is any "system" of treating disease.

Bacterial purity is usually unnecessary except under special circumstances to be discussed in Chapter XIV. Surface water supplies must in any case be treated, before being used for drinking, to avoid the danger of occasional incidental pollution, and the sewage discharged into such waters need only be purified from a bacterial standpoint to such a degree that no undue burden will be placed on the water treatment plant. A judicious use of dilution may often accomplish this end without any refined treatment or disinfection of the sewage itself.

The stipulations in regard to the condition of the effluent from the projected Passaic Valley sewer which were made between the United States government and the Passaic Valley Sewerage Commissioners contained a very cleverly worded statement of the results to be attained by the proposed treatment, which ran as follows:

"1. There will be absence in the New York Bay of visible suspended particles coming from the Passaic Valley sewage.

2. There will be absence of deposits objectionable to the Secretary of War of the United States in the New York Bay coming from the Passaic Valley sewage.

3. There will be absence in the New York Bay and its vicinity of odors due to the putrefaction of organic matters contained in the Passaic Valley sewage thus discharged.

4. There will be a practical absence on the surface of New York Bay of any grease or color due to the discharge of the Passaic Valley sewage at the dispersion area or elsewhere.

5. There will be no injury to the public health which will be occasioned by the discharge from the said sewer into the Bay of New York in the manner proposed, and no public or private nuisance will be created thereby.

6. The absence of injurious effect from said sewage discharge, upon the property of the United States situated in the harbor of New York.

7. The absence of reduction in the dissolved oxygen contents of the waters of New York Bay, resulting from the discharge of Passaic Valley sewage, to such an extent as to interfere with major fish life."

We are not here concerned with the possibility of enforcing broad stipulations of this kind or with their value as legal safeguards; but these clauses certainly outline the general aims of sewage treatment in an illuminating and suggestive way.

Where the purity of drinking water is menaced by a sewage effluent, much higher standards of purity may of course be required. The International Boundary Commission in its report on pollution in the Great Lakes concludes that raw water to be filtered for potable purposes should contain not more than 500 *B. coli* per 100 cubic centimeters as an annual average and should not show *B. coli* in more than half of a series of 0.1 c.c. samples; and sewage treatment should be carried to such a point that water supplies are not polluted beyond this limit.

Sewage treatment in any given case should be carried far enough, — in conjunction with the natural digestive capacity of adjacent waters and with such accessory safeguards as are furnished for example by water purification — to avoid nuisance and danger to health. It should not be carried farther. The student of this subject will do well to bear in mind in this connection as in others, the homely definition of the engineer as a man who can do for one dollar what any fool can do for two.

CHAPTER I

COMPOSITION OF SEWAGE

General Characteristics of Sewage. Sewage may be defined as the water supply of a town or city after it has been used. It is water, polluted by the solid and liquid excreta of the population, household waste, the refuse of various industries, and, in the case of combined systems of sewers, street washings as well. It is water containing mineral, vegetable and animal matter in suspension and in solution.

Besides the mineral and organic matter, sewage contains vast numbers of living organisms — bacteria — twenty-five to fifty million in a liquid ounce; and though by far the greater number of these bacteria are harmless to man and essential agents in the conversion of organic into inorganic matter, there are also present bacteria which are far from harmless, the so-called pathogenic forms; and under certain conditions the destruction of pathogenic bacteria, as well as the removal of the organic matter in sewage, must be considered.

In appearance, sewage, as seen running in the sewers, resembles the dirty water in a washbowl, containing, however, floating on its surface, fecal matter, grease, bits of paper, matches, fruit skins, vegetables, etc. It has only a slight musty odor which is comparatively inoffensive. If placed in a glass cylinder, after being passed through a coarse strainer, and allowed to stand for twenty-four hours, a slimy, greasy matter settles out, but the liquid remains turbid, and offensive odors are given off, caused by the putrefaction of the animal and vegetable matter which it contains. After several weeks the liquid becomes clear, the odor disappears, and the sediment which remains is no longer in a state of putrefaction, having been reduced to a condition similar to that of the humus of the soil.

The change is typical of what always takes place when animal or vegetable matter decomposes in contact with the air, and to bring about this decomposition quickly and inoffensively is the object of all methods of sewage treatment and disposal.

Successful treatment of the sewage of a given community depends largely on the adaptation of the method used to the work required, and this depends not only on the volume of sewage to be treated, but also on the amount and nature of the solid matter. How much of the solid matter is mineral, how much vegetable and animal; how much is in solution in the water, how much in suspension; what decomposition the nitrogenous matter has undergone, how much is still wholly or partially undecomposed; what is the general nature of the undecomposed organic matter, and what substances will be formed during its decomposition: are all questions that must be taken into account in the study of the sewage from a given community.

As a general statement it may be said that the sewage from an American residential town contains in one million parts of sewage from 200 to 800 parts of solid matter (two-tenths to eight-tenths of a gram per liter), or less than one-tenth of one per cent; that about one-half of this solid matter is mineral, the other half vegetable or animal; and that about 75 per cent of the mineral matter and 60 per cent of the vegetable and animal matter is in solution. This can be expressed in tabular form as follows:

Solid matter, 200–800 parts per million.	Mineral, 50%, inoffensive	In solution, 75% In suspension, 25%
	Vegetable and animal, 50%, offensive	In solution, 60% In suspension, 40%

As Ogden and Cleveland express it, " Roughly speaking, the amount of mineral dirt is about one tablespoonful to a barrelful of water, and the combined amount of animal and vegetable matter amounts to another tablespoonful."

Variations in Composition of Sewage. There is a great variation in the amount and character of the solid matter in the sewage flowing in the sewers of different towns and cities. These variations depend on the quantity of water used, the leakage of ground water into the sewers, the quantity and kind of manufacturing waste discharged, and various other factors.

The mineral matter present consists chiefly of sand, clay, iron and aluminium oxides, and the chlorides, carbonates, sulphates and phosphates of the alkalies and alkaline earths. The portion of this mineral matter that is in solution is generally of

little consequence in relation to sewage treatment. The presence of sulphates in large amounts is, however, likely to lead to offensive odors and the hydrogen sulphide formed may have a serious effect upon concrete. The iron salts contributed to many sewages by industrial processes may also cause serious complications. Since the insoluble mineral matter, largely sand and clay, which the sewage may contain in large amounts when street washings are present, is likely to form deposits in streams or clog filters and of course to injure pumps if pumping be necessary, provision must be made for its removal in all methods of sewage treatment.

The character of the organic matter in sewage is of much greater importance, as the nuisances that arise from sewage are due to the anaerobic decomposition of the vegetable and animal substances it contains. The organic substances in sewage can be divided into compounds containing nitrogen and compounds free from nitrogen. The principal nitrogen compounds are urea, proteins, amines and amino acids; the non-nitrogenous compounds are carbohydrates (including cellulose), fats and soap.

Methods of Sewage Analysis. It is not as yet possible by any method of analysis to tell the exact amount of either the nitrogenous or the non-nitrogenous matter present; and in determining the character and concentration of a given sewage, we are obliged to be content with knowing the amount of total and suspended solids; the amount of each that is volatile at a low red heat, often called the organic matter; the amount of fat or grease; the total amount of chlorine as chlorides; the total amount of nitrogen in the organic matter present, called organic nitrogen; the amount of nitrogenous matter that is easily decomposed, as shown by the nitrogen that will be set free as ammonia when the sewage is heated with an alkaline solution of potassium permanganate, called albuminoid nitrogen; the amount of ammonia nitrogen, usually present as ammonium carbonate or sulphate that has been formed by the natural decomposition of the nitrogenous organic matter, now called ammonia nitrogen (free ammonia); the amount of nitrogen combined with oxygen in the form of nitrites and nitrates, which, taken in connection with the ammonia nitrogen, indicates the amount of change that the nitrogenous matter in the sewage has

undergone; and, finally, the amount of oxygen that will be given up from an acid solution of potassium permanganate when sewage is treated with this reagent, which indicates the amount of oxygen that is required to change the organic matter into water, carbon dioxide, sulphur trioxide and nitric acid plus the amount necessary for the oxidation of reducing compounds such as ferrous iron or hydrogen sulphide.

The determination of the above data constitutes what is known as a sewage analysis, and though the knowledge that can thus be obtained is far from being all that could be desired, it yields valuable information, not only in regard to the amount of total solid matter present in the sewage, but also in regard to the amount of organic matter and the character of the nitrogenous substances present.

Prof. Phelps (1905) has shown that by a proper consideration of these analytical results it is possible to form an approximate idea of three distinct characteristics of sewage, namely its *concentration*, or the amount of each of its various constituents; its *composition*, as indicated by the relation between the different concentration values; and its *condition* or degree of decomposition of organic constituents due to age, temperature and initial bacterial content.

Composition of Sewage of Various Cities. It must be clearly understood, that an analysis of a casual sample of any given sewage will give no correct idea as to the character of that sewage, as the sewage of any community usually varies greatly from hour to hour; and it is only from analyses of a large number of samples collected at frequent intervals, or analyses of one or more series of half-hour samples taken during several consecutive days, that any reliable information can be obtained (see Chapter XVII).

A few such analyses of sewage are quoted in the following tables:

TABLE I

ANALYSES OF SEWAGE OF SMALL RURAL COMMUNITIES IN MASSACHUSETTS. CONTAINING LITTLE TRADE WASTE. FLOW LESS THAN 150,000 GALLONS IN 24 HOURS

	Parts per million.			
	Andover.	Stockbridge.	Leicester.	Average.
Inhabitants contributing sewage	3,600	800	500	.....
Total dry weather flow	125,000	75,000	30,000	.....
Flow per person connected with sewer, U. S. G.	35	94	60	63
Total Solids — Total	466.5	235.6	421.7	374.6
Total Solids — Fixed	222.6	147.6	201.2	190.5
Total Solids — Volatile	243.9	88.0	220.5	184.1
Suspended solids — Total	103.3	40.5	140.1	94.6
Suspended solids — Fixed	17.1	9.6	22.8	16.5
Suspended solids — Volatile	86.2	30.9	117.3	78.1
Dissolved solids — Total	363.2	195.1	281.6	279.9
Dissolved solids — Fixed	205.5	138.0	178.4	173.9
Dissolved solids — Volatile	157.7	57.1	103.2	106.0
Nitrogen as — Amm. N.	39.7	9.8	22.0	23.8
Nitrogen as — Alb. N.	5.6	1.6	4.1	3.8
Oxygen consumed in 5 minutes	49.0	15.2	50.8	38.3
Chlorine	70.0	12.8	54.7	45.8

AUTHORITIES (TABLES I-III.)

Tables I-II, Mass., 1904, pp. 305–455. Boston, October, 1905. to June, 1907, Winslow and Phelps, 1907, p. 410. Lawrence, Day sewage, 6 A.M. to 5 P.M., Average 1908, Clark and Gage, 1909, pp. 15 and 17. Columbus, Johnson, 1905, p. 28. Providence, Bugbee, Personal communication, July 12, 1909, Average of four weekly analyses of hourly samples made July 25 to August 22, 1909. Worcester, Fales, Personal communication, August 28, 1909, Average of hourly samples, taken day and night for three years, 1905–1908. Cleveland, Hoffman, Pratt and Hommon, 1914, p. 41. Philadelphia, Phila., 1911, p. 199. Gloversville, Eddy and Vrooman, 1909, p. 59. Akron, Hommon, 1912, p. 9. Brooklyn, Personal communication from W. T. Carpenter, averages August, 1913, to September, 1915, for solids, August to October, 1913, for nitrogen, August 1913, to March. 1915, for oxygen consumed. Milwaukee, Milwaukee, 1915, p. 91. New Haven, general averages East St. sewer, June, 1917, to April, 1918 (alkalinity and chlorine, June to December, 1917, only). Chicago, personal communication from Langdon Pearse, averages for 39th St. sewer, 1909–1914.

TABLE II

ANALYSES OF SEWAGE OF SMALL CITIES IN MASSACHUSETTS, CONTAINING SOME TRADE WASTE. FLOW BETWEEN 250,000 AND 1,250,000 GALLONS IN 24 HOURS

	Parts per million.						
	Brockton.	Framingham.	Gardner.		Marlboro.	Westboro.	Average.
			Gardner.	Templeton.			
Inhabitants contributing sewage	25,000	7,500	3,500	4,500	10,000	3,000	.....
Total dry-weather flow	878,000	652,000	302,000	250,000	1,100,000	282,000	.....
Flow per person connected with sewer, U. S. G.	35	87	86	55	110	94	77.8
Total solids — Total	818.8	587.7	383.7	430.6	448.2	370.1	506.5
Total solids — Fixed	393.6	288.9	154.4	160.19	229.3	157.1	230.7
Total solids — Volatile	425.2	298.8	229.3	269.7	218.9	213	275.8
Suspended solids — Total	194.6	212.4	154	201.8	137.5	183.3	180.6
Suspended solids — Fixed	21.2	50.2	23.6	24.2	21.8	46.4	31.2
Suspended solids — Volatile	173.4	162.2	130.4	177.6	115.7	136.9	149.4
Dissolved solids — Total	624.2	375.3	229.7	228.8	310.7	186.8	325.9
Dissolved solids — Fixed	372.4	238.7	130.8	136.7	207.5	110.7	199.5
Dissolved solids — Volatile	251.8	136.6	98.9	92.1	103.2	76.1	126.4
Nitrogen as — Amm. N	42.9	26.1	20.2	27.3	26.0	13.8	26
Nitrogen as — Alb. N	7.9	6.5	4.9	6.6	4.4	4.5	5.6
Oxygen consumed in 5 minutes	162.7	47.3	49.2	60.3	44.4	35.5	66.6
Chlorine	131.8	69.9	33.8	43.8	59	23.7	60.3

TABLE III

ANALYSES OF SEWAGE FROM SOME OF THE LARGER CITIES IN THE UNITED STATES

(From published reports and personal letters. See footnote, page 18.)

		Parts per million.												
		Boston.	Lawrence.	Columbus.	Providence.	Worcester.	Cleveland.	Milwaukee	New Haven.	Philadelphia.	Brooklyn.	Chicago.	Gloversville.	Akron.
Number of inhabitants connected with sewers		350,000	....	100,000	189,100	127,100	311,600	.....	.....	.....	.....	.....	16,800	75,000
Total dry-weather flow, U. S. G.		85,000,000	....	9,100,000	18,663,000	15,000,000	49,800,000	.....	.....	.....	.....	.....	2,590,000	11,000,000
Flow per capita		245	....	91	99	160.2	160	.....	.....	.....	.....	.....	154	147
Total solids	Total	.....	717	1026	1715	873.5	.....	1109	.....	.....	.....	.....	.....	.....
	Fixed	.....	430	836	994.5	439.2	.....	.....	.....	.....	.....	.....	.....	.....
	Volatile	.....	287	190	720.5	434.3	.....	.....	.....	.....	.....	.....	.....	.....
Suspended solids	Total	135	149	215	397	285.8	252	259	97	189	162	142	406	238
	Fixed	44	36	134	53.5	78	130	.....	34	59	37	53	177	114
	Volatile	91	113	81	343.5	177.8	126	.....	63	130	125	89	229	124
Dissolved solids	Total	.....	568	811	1318	617.7	.....	850	.....	.....	.....	.....	.....	.....
	Fixed	.....	394	702	941	361.2	.....	.....	.....	.....	.....	.....	.....	.....
	Volatile	.....	174	109	377	256.5	.....	.....	.....	.....	.....	.....	.....	.....
Nitrogen as	Ammonia N	13.9	41.7	11.5	18.4	17.13	12	15.6	4.4	4	32	9.2	12	6.6
	Total alb. N	.....	6.6	.....	8.86	7.33	.....	8.9	.....	.....	.....	.....	.....	.....
	Dis. alb. N	.....	3.4	.....	4.28	3.34	.....	.....	.....	.....	.....	.....	.....	.....
	Sus. alb. N	.....	3.2	.....	4.58	3.99	.....	.....	.....	.....	.....	.....	.....	.....
	Total organic N.	9.1	....	9.7	.....	.....	8.5	32.9	.....	6.3	27	7.6	23	10.6
	Dis. organic N.	5.8	....	3.6	.....	.....	.....	.....	.....	.....	18	.....	.....	6.9
	Nitrates	.....	....	.....	.....	.....	.....	0.57	1.6	.....	.....	0.32	.....	.....
Oxygen consumed	Total	56	55.7	56	93.6	120.7	50	125	.....	76	64	42	95	86
	Dissolved	43	....	28	46.2	.....	.....	.....	.....	.....	42	.....	.....	.....
Chlorine		.....	143	67	540.5*	114	254	194	1062*	39	.....	39	158	917
Dissolved oxygen		.....	....	.....	.....	.....	7.2	2.2	.....	.....	.....	1.7	.....	.....
Alkalinity		.....	....	.....	.....	.....	137	259	62	.....	.....	209	.....	.....

* Contains sea water. In New Haven case the sea water was condenser water from a certain factory. When the factory was shut down for four months the chlorine fell to 53 and the alkalinity to 37.

TABLE IV

ANALYSES OF SEWAGE OF ENGLISH CITIES CONTAINING TRADE WASTE

(From published reports and personal letters.)

	Parts per million.						
	Birmingham.	Bradford.	Leeds.	Leicester.	Manchester.	Sheffield.	Average.
Number of inhabitants connected with sewers	900,000	240,000	454,450	235,000	575,000	400,000	.....
Total dry-weather flow, U. S. G.	32,544,000	15,600,000	20,648,400	11,551,244	30,360,000	19,200,000	.....
Flow per capita, U. S. G.	36	65	45.4	47.4	52.8	48	49.1
Total solids — Total	1,947	2,650	1,843	1,805	1,235	.....	1,896
Total solids — Fixed	.....	1,774	.....	.....	592	.....	.....
Total solids — Volatile	.....	876	.....	.....	643	.....	.....
Suspended solids — Total	718	840	775	680.3	327	417.5	668
Suspended solids — Fixed	.....	264	.....	.....	163	170.7	.....
Suspended solids — Volatile	.....	576	.....	.....	164	246.8	.....
Dissolved solids — Total	1,229	1,810	1,069	1,124.7	908	.....	1,228
Dissolved solids — Fixed	.....	1,510	.....	869.2	429	.....	.....
Dissolved solids — Volatile	.....	300	.....	255.5	479	.....	.....
Nitrogen as — Amm. N	41.6	47.8	22.8	45.064	29.3	32.18	36.46
Nitrogen as — Alb. N	16.2	32.8	7.88	15.6	7	9.02	14.75
Oxygen consumed in 4 hrs. at 80°F. — Total	259.2	202.3	99	125.22	113.6	79.6	146.5
Oxygen consumed in 4 hrs. at 80°F. — Dissolved	138.9	.....	.....	.....	.....	.....	.....
Chlorine	205	149	165	127.2	200	124.3	161.8

Birmingham: Watson. Birmingham Sewage Disposal Works, Proc. Inst. of Civil Engineers, 1910. Average for 1901-1905.
Bradford: Garfield. Personal communication, August, 1909. General average.
Leeds: Hart. Personal communication, July, 1909. Average of 23 analyses.
Leicester: Mawbey. Personal communication, September, 1909. Average of 50 analyses, loss on ignition, 25.
Manchester: Fowler. Personal communication, August, 1909. Average for 1905.
Sheffield: Wike. Personal communication, August, 1909. Average of 130 analyses.

The above tables give a general idea of the nature of sewage, and show that the amount of total solid matter, organic matter and nitrogenous matter varies within wide limits in the sewage of different communities; that the sewage of a large city is very different from the sewage of a rural community; and that the sewage of English cities (the same is true of Continental cities) contains more solid matter in a given volume, or, as usually stated, is more concentrated, than the sewage of American cities.

Calculation of Sewage Constituents in Grams per Capita. Analyses of sewage like those given are also used to obtain tables known as Grams per Capita tables, by reducing the results of analysis and the gaugings of the sewage flow to a basis of amount in weight of the various constituents per capita per day. For this purpose the following formula can be used:

$$\text{Grams per capita per day} = A \times C \times 0.001 \times 3.785,$$

when A = parts per million; C = gallons per capita per day; 3.785 = number of liters in a U. S. gallon.

Such tables may be of some assistance in forming a judgment of the relation between the number of people served by a sewer system and the probable quality of the sewage. It is not safe, however, to assume that such a calculation, based upon the data ordinarily available, even when taken from several places and averaged, will apply with reasonable accuracy to other cities. This is clearly demonstrated by the wide variations in Tables V and VI, compiled from the data given in Tables I and II.

These towns are all sewered on the separate system and the sewage is what would be classed as domestic sewage, although there are considerable amounts of industrial waste in the sewage of Brockton, and small amounts in the sewage of some of the other towns.

The variations are no doubt partly due to errors in sampling the sewage, measuring the flow, and estimating the population connected with the sewer systems. The quantity and character of ground water in the sewage has a marked influence upon such data, as does also the surface water from streets and areas, which not infrequently finds its way into "separate" sewers, through cracks in the sewer or through drain connections,

— regulations to the contrary notwithstanding. Such connections, made from time to time, are seldom recorded and are soon forgotten, though the ground water which enters through them may have a very marked influence upon the composition of the sewage.

The Problem of Manufactural Wastes. Manufactural wastes are extremely variable in quality, as well as in proportion to the quantity of domestic sewage. In many cities the various ingredients usually determined may come as largely from industrial works as from dwellings. It is obvious, therefore, that there is no logical relation between the impurities in sewage from an industrial city and the population served by the sewer system. This is well shown in Tables VII and VIII which have been calculated from data in Tables III and IV.

TABLE V

QUANTITIES OF PRINCIPAL CONSTITUENTS IN GRAMS PER CAPITA DAILY. SEWAGE OF SMALL RURAL COMMUNITIES IN MASSACHUSETTS, CONTAINING LITTLE TRADE WASTE

(Calculated from table on page 18.)

	Grams per capita.			
	Andover.	Stock-bridge.	Leicester.	Average.
Flow per person connected with sewer, U. S. G.	35	94	60	63
Total solids — Total	62	84	96	89
Total solids — Fixed	30	53	46	45
Total solids — Volatile	32	31	50	44
Suspended solids — Total	14	14	32	23
Suspended solids — Fixed	2	3	5	4
Suspended solids — Volatile	12	11	27	19
Dissolved solids — Total	48	70	64	67
Dissolved solids — Fixed	27	50	41	42
Dissolved solids — Volatile	21	20	23	25
Nitrogen as Ammonia N.	5.3	3.5	5	5.68
Nitrogen as Alb. N.	0.7	0.6	0.9	0.91
Oxygen consumed in 5 minutes	6.5	5.4	11.5	9.14
Chlorine	9.0	4.6	12.0	10.92

The great influence of industrial wastes under extreme conditions is well illustrated by comparing the composition of two Chicago sewages, that of the 39th St. sewer which carries an ordinary rather weak domestic sewage, and the day sewage from the Center Ave. sewer which receives most of the wastes

from the Stockyards District. The data are taken from the Report on Industrial Wastes from the Stockyards and Packingtown in Chicago, made to the Board of Trustees of the Sanitary District of Chicago, October, 1914.

	Parts per million.					
	Organic N.	Ammonia N.	Nitrites.	Nitrates.	Oxygen consumed.	Chlorine.
39th St.....	7.8	9.1	0.10	0.33	43	40
Center Ave.* ..	79	22	0.49	3.04	268	1100

	Total suspended solids.	Volatile suspended solids.	Alkalinity.	Fats.
39th St.....	144	90	212	23
Center Ave.*.....	605	461	291	198

* Day sewage only.

TABLE VI

QUANTITIES OF PRINCIPAL CONSTITUENTS IN GRAMS PER CAPITA DAILY. SEWAGE OF SMALL CITIES IN MASSACHUSETTS, CONTAINING SOME TRADE WASTE

(Calculated from table on page 19.)

	Grams per capita.						
	Brockton.	Framingham	Gardner.		Marlboro.	Westboro.	Average.
			Gardner.	Templeton.			
Flow per person connected with sewer, U. S. G.....	35	87	86	55	110	94	77.8
Total solids { Total.....	115	194	125	90	187	132	149
Total solids { Fixed.....	43	95	50	34	96	56	68
Total solids { Volatile.....	72	99	75	56	91	76	81
Suspended solids { Total.....	55	70	50	42	57	66	53
Suspended solids { Fixed.....	7	16	8	5	9	17	9
Suspended solids { Volatile.....	48	54	42	37	48	49	44
Dissolved solids { Total.....	60	123	75	48	130	66	96
Dissolved solids { Fixed.....	36	78	42	29	87	39	59
Dissolved solids { Volatile.....	24	45	33	19	43	27	37
Nitrogen as { Amm. N....	4.2	8.6	6.6	5.7	10.8	4.9	7.7
Nitrogen as { Alb. N.....	1.2	2.1	1.6	0.14	1.8	1.6	1.6
Oxygen consumed in 5 minutes.....	21	16	16	13	19	13	19.6
Chlorine.....	13	23	11	9	25	8	17.7

TABLE VII

QUANTITIES OF PRINCIPAL CONSTITUENTS IN GRAMS PER CAPITA DAILY. SEWAGE OF ENGLISH CITIES, CONTAINING TRADE WASTE

(Calculated from table on page 21.)

	Grams per Capita.					
	Birmingham.	Bradford.	Leeds.	Leicester.	Manchester	Sheffield.
Flow per person connected with sewer, U. S. G.	36	65	45.4	47.4	52.8	48
Total solids — Total	266	652	317	326	247	....
Total solids — Fixed	....	437	....	....	118	....
Total solids — Volatile	....	215	....	....	129	....
Suspended solids — Total	98	207	133	123	65	76
Suspended solids — Fixed	....	65	....	....	32	31
Suspended solids — Volatile	....	142	....	....	33	45
Dissolved solids — Total	168	445	184	203	182	....
Dissolved solids — Fixed	....	372	....	157	86	....
Dissolved solids — Volatile	....	73	....	46	96	....
Nitrogen as — Amm. N.	5.68	11.7	3.9	8.3	5.8	5.8
Nitrogen as — Alb. N.	2.21	8.2	1.4	2.8	1.4	1.64
Oxygen consumed in 4 hrs. — Unfiltered	35.3	49.8	17.0	22.1	22.7	14.5
Oxygen consumed in 4 hrs. — Filtered	18.9	....	....	....	....	....
Chlorine	28	36.7	28.4	23	40	22.6

TABLE VIII

QUANTITIES OF PRINCIPAL CONSTITUENTS IN GRAMS PER CAPITA DAILY. SEWAGE OF LARGER AMERICAN CITIES CONTAINING TRADE WASTES

(Calculated from table on page 20.)

	Grams per capita.						
	Boston.	Columbus.	Providence.	Worcester.	Cleveland.	Akron.	Chicago.*
Flow per capita, U. S. G.	245	91	99	118	160	147	166
Total solids — Total	....	353	642	390	....	....	....
Total solids — Fixed	....	288	372	196	....	....	....
Total solids — Volatile	....	65	270	194	....	....	....
Suspended solids — Total	125	74	149	114	152	132	145
Suspended solids — Fixed	41	46	20	35	79	63	54
Suspended solids — Volatile	84	28	128	80	74	69	91
Dissolved solids — Total	....	287	494	276	....	....	....
Dissolved solids — Fixed	....	242	352	161	....	....	....
Dissolved solids — Volatile	....	45	142	115	....	....	....
Nitrogen as — Ammonia N.	13	4	7	8	7	4	9
Nitrogen as — Total alb. N.	....	....	3	3	....	....	....
Nitrogen as — Dis. alb. N.	....	....	2	1	....	....	....
Nitrogen as — Total org. N.	8	3	....	....	5	6	8
Nitrogen as — Dis. org. N.	5	1	....	....	....	4	....
Oxygen consumed — Total	52	19	35	54	33	48	43
Oxygen consumed — Dissolved	40	10	17	....	....	....	....
Chlorine	....	23	230	51	154	375	40

* Chicago data for 1909–1913 furnished by Langdon Pearse.

Suspended Matter in Sewage. The suspended solid matter in sewage, particularly that fraction which is of organic nature, is of very special importance. It is frequently the accumulation of deposited solid materials in stream beds and along foreshores that makes sewage treatment imperative, and when treatment is actually undertaken it is often the suspended solid matter which offers the most serious difficulties.

The amount of suspended solid matter, as indicated by the tables given above, varies in the sewage of American cities from 40 to over 400 parts per million, or from 14 to over 150 grams per capita, of which from one-half to two-thirds is of organic nature. Fuller (1912) gives the following tabulated review of the suspended solids of sewages calculated in several different ways.

TABLE IX

TOTAL SUSPENDED MATTER IN VARIOUS SEWAGES

(Fuller, 1912.)

Place.	Parts per million.	Grams per capita.	Tons per million U. S. gallons.	Tons per 1000 population per year.
Plainfield, N. J.	173	60	0.72	24
Framingham, Mass.	212	70	0.88	28
Boston, Mass.	135	168	0.56	67
Gardner, Mass.	154	50	0.64	20
Marlboro, Mass.	137	57	0.57	23
Brockton, Mass.	195	55	0.81	22
Worcester, Mass.	256	175	1.06	70
Providence, R. I.	397	149	1.65	60
Columbus, O.	215	98	0.90	39
Chicago, Ill.	141	155	0.59	62
London (north), England	483	87	2.00	35
London (south), England	408	92	1.70	37
Huddersfield, England	346	142	1.44	57
Leeds, England	610	137	2.54	55
Leicester, England	635	143	2.64	57
Manchester, England	370	102	1.54	41
Birmingham, England	718	98	2.98	39
Bradford, England	840	207	3.50	83
Sheffield, England	417	76	1.73	31

The ordinary distinction between suspended solids and dissolved solids, which depends on the separating action of filter paper or an asbestos mat, is a somewhat fallacious one from a practical standpoint. Of the solids taken out by filtration a considerable fraction is in a semi-colloidal state and will not settle out on standing for any reasonable period. Metcalf and

Eddy (1915) give the following hypothetical estimate of the subdivisions of the total solids in an average American sewage.

TABLE X

DISTRIBUTION OF SOLID MATERIAL IN AN AVERAGE SEWAGE ACCORDING TO PHYSICAL CONDITION

(Metcalf and Eddy, 1915.)

Total solids, 800	Suspended solids, 300	Settling solids (2 hours) 150	Organic...	100
			Mineral....	50
		Suspended colloidal solids, 150	Organic...	100
			Mineral...	50
	Dissolved solids, 500	Dissolved colloidal solids, 50	Organic...	40
			Mineral...	10
		Dissolved crystalloidal solids, 450	Organic...	160
			Mineral...	290

Organic Constituents of Sewage. It is the organic constituents, making up on the average perhaps two-thirds of the matter in suspension and half of the matter in solution, which constitute the chief problem in sewage treatment. In the study of these constituents we ordinarily rely on a few simple determinations each of which indicates a group of widely diverse substances. These determinations — for total organic nitrogen, albuminoid nitrogen (easily decomposed nitrogenous matter), ammonia nitrogen (nitrogenous matter that has already been decomposed), nitrite and nitrate nitrogen (oxidized nitrogenous matter), and oxygen consumed (oxygen necessary to oxidize a certain proportion of the carbonaceous matter) — will be discussed more fully in Chapter XVII. At this point it is desirable to refer very briefly to some of the principal substances actually present, the amount and condition of which is indirectly indicated by the determinations of various forms of nitrogen used in sewage analysis.

As has been previously stated, the organic matter in sewage is made up chiefly of urea, proteins and their decomposition products, carbohydrates (including cellulose and woody fiber), fats and soap. These substances, chiefly through the action of bacteria, undergo a more or less active decomposition, and to follow the changes which take place it is necessary to have a knowledge of the composition and properties of the materials concerned.

The amount of the various organic materials in an average American sewage has been estimated by Winslow and Phelps (1906) to be about as indicated below. It is assumed that the sewage in question contains 400 parts per million of organic matter, half in solution and half in suspension.

TABLE XI

ORGANIC CONSTITUENTS IN AN IDEAL AMERICAN SEWAGE

(Winslow and Phelps, 1906.)

	Parts per million.			
	Nitrogen.	Carbon.	Other elements (hydrogen, oxygen, sulphur, phosphorus, etc.).	Total.
Nitrogenous matter	15	75	60	150
Fats	.....	35	15	50
Carbohydrates	.....	90	110	200
Total	15	200	185	400

Urea and the Proteins. Urea, the chief constituent of urine, is a compound of carbon, hydrogen, oxygen and nitrogen, having the formula $CO(NH_2)_2$. It is a white crystalline substance, soluble in water, and is readily converted into ammonium carbonate; so quickly does this change take place that undecomposed urea is rarely found in sewage at an outfall sewer. The odor of ammonia that so often pervades the air in the neighborhood of urinals and barnyards is chiefly due to the decomposition of this substance.

The proteins form the principal constituents of the animal organism. They are also found in the living parts of plants, particularly in the seeds. The number of known proteins is large, and they are sometimes divided into groups according to their animal or vegetable origin; but between the members of the different groups there are no very striking chemical differences. They all contain carbon, oxygen, hydrogen and nitrogen; many contain sulphur, some iron and phosphorus. The distinguishing characteristic of the proteins is the fact that they all contain a fairly constant proportion of nitrogen — between 15 and 16 per cent.

In their coagulated state proteins are white, amorphous substances, some of which, like egg albumin, are soluble in water,

while others, like globulin, are insoluble. Most of them are soluble in dilute mineral acids; all are soluble in concentrated acetic and phosphoric acids. In chemical composition they differ very slightly, as is shown by the percentage composition of three of the chief proteins:

	Albumin.	Fibrin.	Casein.
Carbon	53.5	52.7	53.8
Hydrogen	7.0	6.9	7.2
Nitrogen	15.5	15.4	15.6
Oxygen	22.4	23.8	22.5
Sulphur	1.9	1.2	0.9

The molecular structure of proteins is highly complex, the number of atoms in a single molecule being generally considered to reach into the thousands. They are supposed to be synthesized by the linking of amino acids, the simplest of which is amino acetic acid $CH_2 \begin{matrix} / CO_2H \\ \backslash NH_2 \end{matrix}$.

The Carbohydrates. The term " carbohydrate " is applied to a large class of compounds widely distributed in nature. The molecule of the common carbohydrates contains six, or a multiple of six, carbon atoms; and the ratio of their hydrogen to their oxygen atoms is the same as that of these elements in water, two of hydrogen to one of oxygen. The carbohydrates include starches, sugars, cellulose and wood fiber. The general composition of these compounds is shown by the formulæ:

Starches, $(C_6H_{10}O_5)_x$.

Sugar, { Glucose, $C_6H_{12}O_6$. / Sucrose, $C_{12}H_{22}O_{11}$.

Cellulose, $(C_6H_{10}O_5)_y$.

Some carbohydrates, like sugars, are soluble in water; others, like the starches, are insoluble. Starches and cellulose can be converted into sugars containing six carbon atoms by dilute acids and by certain organic ferments. The sugars can be fermented into alcohol by yeast and certain enzymes, carbon dioxide being evolved.

The Fats and Soaps. The fats are compounds of the triatomic alcohol, glycerol, combined with fatty acids, such as stearic, pal-

mitic and oleic acids. These compounds are known as glycerides. The special names of the most important fats are stearin, palmitin, olein and butyrin, and their chemical composition is as follows:

Stearin, $C_3H_5(C_{18}H_{35}O_2)_3$.
Olein, $C_3H_5(C_{18}H_{33}O_2)_3$.
Palmitin, $C_3H_5(C_{16}H_{31}O_2)_3$.
Butyrin, $C_3H_5(C_4H_9O_2)_3$.

Most animal fats are a mixture of two or more of these substances in different proportions; and some, as, for example, butter, contain glycerides of simpler fatty acids.

Of the most important glycerides, stearin and palmitin are solids, olein and butyrin liquids. They are insoluble in water, but soluble in alcohol. Chemically they consist of carbon, hydrogen and oxygen, the percentage composition depending upon the particular kind of fat. Their molecular structure is not complicated, but they are among the more stable of organic compounds, and, outside of the human body, are not easily broken down by bacteria. They are, however, acted upon by mineral acids, giving glycerol and a fatty organic acid; and they are decomposed by the alkalies into glycerol and the alkali salts of the fatty acids, compounds known under the name of soaps.

Soap is the generic name given to the mineral salts of the fatty acids; and soaps are formed when fatty acids or fats are treated with basic hydroxides, a process known as saponification. The most common soaps, whether hard or soft, are to-day made from sodium hydroxide. Such soaps, being universally used, occur in solution and suspension in considerable quantities in alkaline sewage, and, being only slightly acted upon by bacteria, undergo very slight change as long as the sewage remains alkaline. When the sewage becomes acid, they are decomposed into insoluble fatty acids and soluble salts of sodium — or potassium in the case of old-fashioned soft soap.

The amount of fat present in sewage is of considerable importance, because of the clogging of filters, etc., which sometimes results when these substances are present in excess. On the other hand, if a recently devised process for extracting grease from sewage proves to be practical, the grease in sewage may have a potential commercial value. The estimate of the amount of these substances cited on page 28 is probably somewhat excessive.

The amount of fats in the sewage of Waterbury, Conn., Columbus, Ohio, Philadelphia, Pa., and Chicago, Ill., varied between 20 and 30 parts per million, Gloversville, N. Y., with a heavy burden of industrial wastes, alone reaching 48 parts (from references cited on page 18). In connection with his studies on the treatment of Boston sewage with sulphurous acid Weston (1916) reports that the Boston sewage examined contained 36.8 parts per million of grease, while after the acid treatment (which converts some of the soaps to fatty acids) the amount extracted rose to 51.7 parts.

The Bacteria in Sewage. The bacteria in sewage are significant from several different standpoints. Certain members of this group of micro-organisms are active in the ordinary decompositions which produce foul odors in the putrefaction of sewage materials, and are utilized in a practical way for the liquefaction and gasification of sludge in the septic and Imhoff tank. It is this type of bacteria which makes up the greater part of the flora developing on ordinary gelatin or agar plates. The nitrifying bacteria which play a principal rôle in the purification of sewage by oxidation will not grow in the ordinary laboratory media and are thus ignored in routine work. The count obtained on ordinary plates is often called the " Total Count," but this term is inaccurate since any method of bacterial cultivation we may use will only permit the development of those types of organisms for which the food material, temperature of incubation and other conditions are suitable. In general then the agar or gelatin count at 20 degrees represents the great mass of miscellaneous saprophytic bacteria including the putrefactive forms; and the count will roughly vary with the strength of the sewage and with its staleness. The count obtained by incubation at 37° C. will always be lower than that obtained at 20 degrees but the difference with sewage is much less than in the case of pure water.

The types of bacteria characteristic of the intestine are of particular significance where the protection of water supplies, bathing grounds or shellfish layings is involved. As an index of these types we use the *Bacillus coli* group and in sewage work it is common to include under this general term all forms which produce gas from sugar media, although this test is only a rough approximation to the truth.

The general range of variation in the bacterial content of sewages may be indicated by the following quotation from Prescott and Winslow's Elements of Water Bacteriology.

"The total number of bacteria and the number of colon bacilli naturally vary widely in the sewages of different cities and towns. European sewages, being more concentrated, show as a rule higher numbers than are found in America. Results compiled from various sources show from 1,000,000 to 5,000,000 bacteria in the sewages of Essen, Berlin, Charlottenburg, Leeds, Exeter, Chorley and Oxford; 2,000,000 to 10,000,000 in the sewages of London, Walton, and W. Derby; and over 10,000,000 in the sewages of Paris, Ballater and Belfast (Winslow, 1905). The number of colon bacilli in English sewages varies from 50,000 to 750,000. In American sewages, on the other hand, bacteria are somewhat less numerous. At Lawrence the determinations made from 1894 to 1901 showed on the average 2,800,000 bacteria per c.c. At Worcester, Eddy reported 2,713,000 in 1901 (Eddy, 1902); at Ames, Iowa, Walker (1901) found 1,248,256 in the same year. At Columbus, Johnson (1905) reports an average of 3,600,000 bacteria per c.c.; the individual numbers varied from 320,000 to 27,000,000. The number of colon bacilli varied from 50,000 to 100,000 and averaged 500,000. Day samples of Boston sewage collected three times a week, from October, 1906, to April, 1907, showed an average of 1,200,000 bacteria per c.c. In summer months numbers are notably higher than at other seasons in many sewages. Thus in 1903, Boston sewage contained 2,995,000 bacteria in July; 4,263,600 in August; 11,487,500 in September; 3,693,000 in October; 587,100 in November; and 712,000 in December (Winslow, 1905). There is also a marked diurnal variation in the bacterial content of sewage, since the flow contains a smaller proportion of intestinal matter at night than at other times." For example, a series of hourly samples at the Sewage Experiment Station of the Massachusetts Institute of Technology showed counts varying from 4,600,000 bacteria per c.c. between 3.30 and 7.30 P.M. to 400,000 between 3.30 and 7.30 A.M.

Occasionally very peculiar results will be recorded as a result of the presence of antiseptic industrial wastes. Thus at New Haven, Conn., it was found that sewage samples collected on Sundays and at 8 A.M. on week days contained an average of

1,890,000 bacteria per c.c. and 193,000 colon bacilli, while samples collected during the daytime on week days contained only 136,000 bacteria and only 5150 colon bacilli per c.c.

The reason for this condition was found to be the presence of copper salts introduced in the wastes from the washing of cartridges in a large munition factory. Above this establishment the sewage contained on one day 990,000 bacteria per c.c. and no copper; below the factory connections it contained only 3000 bacteria and 8.8 parts per million of copper. It may be noted incidentally that the presence of this antiseptic interfered very seriously with the operation of various biological processes of sewage treatment, such as the Imhoff tank and the activated sludge process.

The Biolysis of Sewage. The term "biolysis of sewage" is used to express the breaking down, or the decomposition, of animal and vegetable substances. Such decomposition is usually divided into two stages, the first called putrefaction, and the second or final stage, oxidation or nitrification. This decomposition of organic matter is chiefly brought about by bacteria, or by substances formed by bacteria, the so-called enzymes. In the first or putrefactive stage, the active agents are mainly bacteria which live and multiply in the absence of air. In the second stage the active agents are bacteria which require oxygen.

	Substances dealt with.	Characteristic products.
Initial.		
Transient aerobic changes by the oxygen of the water supply rapidly passing to —	Urea, ammonia, and easily decomposable matters.	
First Stage.		
Anaerobic liquefaction and preparation by hydrolysis.	Albuminous matters. Cellulose and fiber. Fats.	Soluble nitrogenous compounds. Phenol derivatives. Gases. Ammonia.
Second Stage.		
Semi-anaerobic breaking down of the intermediate dissolved bodies.	Amino-compounds. Fatty acids. Dissolved residues. Phenolic bodies.	Ammonia. Nitrites. Gases.
Third Stage.		
Complete aeration: oxidation and nitrification.	Ammonia and carbonaceous residues.	CO_2, H_2O, and nitrates.

Rideal, who has made a very careful study of the changes that take place during biolysis, is responsible for the division of the process into the four stages cited (Rideal, 1906).

Taking Rideal's summary as an outline of what takes place, the changes that are thus brought about by the action of micro-organisms on proteins, carbohydrates and fats, may be considered a little more in detail.

1. Proteins. In the first of Rideal's stages, the proteins are broken down into the albumoses and peptones, with the separation of the sulphur as hydrogen sulphide, or as mercaptans, sulphur alcohols having very disagreeable odors. The albumoses and peptones have a less complex molecular structure than the proteins; they are soluble in water and are not coagulated by heat. These compounds, also during the first stage, break down into the so-called amino acids, chiefly acids of the fatty hydrocarbon series containing carbon, hydrogen, oxygen and nitrogen in which one hydrogen atom is replaced by the amino group, (NH_2).

The amino acids thus formed are decomposed during the first and second of Rideal's stages, giving ammonia, phenols, fatty and aromatic acids. All of the nitrogen, however, is not converted into ammonia, for part remains united to hydrogen and carbon, forming amines like trimethyl amine, $(CH_3)_3N$, part is liberated as nitrogen, while a certain portion is undoubtedly converted directly into nitrous acid.

The last change in the process of decomposition is a partial or complete oxidation of the organic substances formed by the decomposition of the amino acids, resulting in the production of water, carbon dioxide, nitrous and nitric acids. Gaseous nitrogen is sometimes liberated in large quantities by the action of amines on nitrous acid.

The principal products of the bacterial action on the proteins are amino acids of the fatty hydrocarbon series, relatively small amounts of the aromatic compounds, such as phenyl alanin, tyrosin, tryptophan, phenol, skatol and indol, also being formed.

2. Carbohydrates. Sugar and starches are very easily hydrolized and broken down by the action of bacteria, and though alcohol may be one of the products of the decomposition, the principal substances formed seem to be butyric acid, lactic acid, water, carbon dioxide and hydrogen. Cellulose and woody

fiber are also broken down and liquefied, but comparatively slowly, and the action is often so sluggish that they are but little changed in the process of sewage treatment. Hydrolysis plays an important part, at least during the first stages of the decomposition of cellulose, and the products formed are undoubtedly similar to those produced from the sugars and starches.

3. Fats. The decomposition of fats, brought about by the action of bacteria and molds, is again largely a process of hydrolysis, the fatty acids, stearic, palmitic, oleic, butyric, and glycerol, being the chief products. Further hydrolysis converts the fatty acids into carbon dioxide, hydrogen and methane. The breaking down of the fats takes place very much more slowly than that of the proteins, and it appears probable that they must first emulsify before being acted upon by bacteria. This emulsification is at least partly brought about by the ammonia set free in the decomposition of the amino acids. The slow decomposition of fats and greases makes a sewage containing large amounts of these substances very much more difficult to purify than a sewage in which the organic matter is mainly in the form of proteins and carbohydrates.

In the decomposition of proteins, carbohydrates and fats, fatty acids, as has been stated, are always formed, and these acids are further broken down, giving as final products water, carbon dioxide, hydrogen and methane. The table on page 36 taken from Rideal (1906), gives in a concise form the products formed by the decomposition of the principal fatty organic acids.

The propionic, butyric, succinic and valeric acids under active bacterial action, as well as any amino acids which are formed as products of the decomposition of the salts of the more complex acids, are also eventually broken down, giving, as the final product, methane, hydrogen, carbon dioxide, ammonia and water.

The processes of Rideal's first two stages (see p. 33) will manifest themselves as follows in the results of the determinations ordinarily made in sewage analysis Dissolved oxygen, nitrates and nitrites will soon disappear, ammonia nitrogen will rapidly and progressively increase, albuminoid nitrogen will decrease slightly and total organic nitrogen (particularly that portion which is in solution) will markedly decrease, as will the values for oxygen consumed.

TABLE OF FERMENTATION OF ORGANIC SALTS

(For simplicity, the sodium salts are taken, though the lime salts are rather more fermentable.)

Salt fermented.	Products.
Formate	Acid sodium carbonate, $NaHCO_3$, carbonic acid and hydrogen.
Acetate	Acid sodium carbonate, $NaHCO_3$, carbonic acid, and methane, CH_4.
Lactate. Undergoes four different fermentations.	1. Propionic acid, and as by-products, acetic and succinic acids, and alcohol. 2. Propionic and valeric acid. 3. Butyric and propionic acid. 4. Butyric acid and hydrogen.
Malate. Different fermentations.	1. Chief product, propionic acid; by-product, acetic acid. 2. Chief product, succinic acid; by-product, acetic acid. 3. Butyric acid and hydrogen. 4. Lactic acid and CO_2.
Tartrate	1. Chief product, propionic acid; by-product, acetic acid. 2. Butyric acid. 3. Chief product, an acetate; by-products, alcohol, butyric and succinic acids.
Citrate	Acetic acid in large quantities, with small quantities of alcohol and succinic acid.
Glycerate	1. An acetate, with small quantities of succinic acid and alcohol. 2. Formic acid, with some methyl alcohol and acetic acid.

In Rideal's third stage (nitrification) nitrites will re-appear and then nitrates. In the intermittent sand filter the conversion of ammonia nitrogen to nitrate nitrogen may be almost quantitatively complete, while in the trickling filter a considerable proportion of the nitrogen is changed not to nitrates but to a stable humus-like organic form. In the contact bed much of the nitrogen is given off in gaseous form into the air, while in activated sludge treatment atmospheric nitrogen is often actually absorbed and stored.

These statements regarding the biolysis of sewage can only be regarded as general, since many other substances besides

those noted are undoubtedly formed. During the decomposition of proteins the odors which are given off are not due alone to hydrogen sulphide, the mercaptans and the amines, but also to other substances regarding which we have practically no knowledge. Further, in the changes that have been noted, there is always formed a comparatively large amount of stable organic matter which most effectively resists further change, resembling in its properties the humus of the soil. What this substance is, or how it is produced, we have little, if any, idea.

Though much remains to be worked out before we can state exactly what takes place during the biolysis of sewage, the essential facts are as follows: urea is decomposed, ammonium carbonate being formed; the proteins are first broken up into peptones, then into amino acids, these amino acids being resolved into nitrogen, amines, ammonia and the fatty acids; the fats yield glycerol and fatty acids; the carbohydrates yield chiefly acids like lactic and butyric, though some alcohol may be formed; and the fatty acids are to a greater or less extent decomposed into hydrogen, methane gas, carbon dioxide and water. The sulphur of the proteins is changed into hydrogen sulphide or into mercaptans, and the amines and ammonia set free from the amino acids are oxidized to nitrous and nitric acid. During these changes a certain amount of stable organic matter is formed, similar to the humus of the soil.

The important question in sewage treatment is, — How can these changes be brought about without causing offense, and at a minimum cost?

CHAPTER II

DISPOSAL OF SEWAGE BY DILUTION

The Disposal of Sewage by Discharge into Water. Direct discharge into the nearest body of water is still the commonest method of sewage disposal in many sections of the United States; and it proves in many cases a fairly satisfactory one except for cities located on small rivers. Where the body of diluting water is of sufficient size, the sewage disappears without nuisance and without permanent injury to the lake or stream. Thus Weston estimates that the Mississippi River above New Orleans receives a billion and a half gallons of sewage per day, amounting to a flow of 2310 cubic feet per second. Yet at New Orleans the river is no more polluted than the average surface stream.

This method of dealing with sewage is commonly called " Disposal by Dilution," and the term is a fairly descriptive one. Impurities are of course just as truly present whether they are distributed in a large body of water or a small one. From a practical standpoint, however, dilution amounts to the same thing as removal, and may reasonably be considered to constitute real purification. If a small stream which contains one typhoid germ in every tumblerful of water mixes with an unpolluted stream of a hundred times its volume, the danger of contracting typhoid fever by drinking a glass of water is diminished to one per cent of what it was before.

Beyond the effect of dilution, pure and simple, there are other processes of purification at work in a stream, which ultimately lead to the oxidation of unstable organic compounds and the destruction of pathogenic bacteria. These processes of chemical and bacterial self-purification are quite distinct from each other and must be considered separately. Both, however, are indirectly favored by dilution; and dilution itself, aside from any chemical or biological agencies, often plays a predominant part. In order to be convinced of this it is only necessary to compare the reduction of chlorine with the diminution of bacteria

and organic matter in a stream which is undergoing self-purification. It will generally appear that nine-tenths of the improvement manifest is due to dilution alone.

Self-Purification of Streams. There are two principal factors which mainly control the removal of organic polluting material from water — the direct physical effect of sedimentation and the chemical changes in the organic matter itself, which are chiefly produced by the action of aerobic bacteria. The primary force at work in freeing the water from suspended impurities is no doubt the force of gravity, and this alone accounts for a large part of the purification which ensues in streams or ponds. Gravity alone, however, if unaccompanied by any other action, would only localize the trouble on the bottom, and might even accentuate it by the very fact of concentration. Where suspended impurities gather too rapidly, sludge banks often accumulate to considerable dimensions, and instead of self-purification a gross nuisance may result.

True organic self-purification implies chemical changes of such a nature as to alter the decomposable organic matter to a stable form. This is brought about by a number of agencies. Direct chemical changes play a certain part, as, for example, in the simplification and oxidation of sulphur and iron compounds. According to Adeney (Letts and Adeney, 1908) such direct chemical oxidations are of very considerable significance. Algæ also play a part. Bokorny (1897) and various observers attribute much importance to the activity of the Green Algæ; but Chick and others (Brezina, 1906) have shown that while these organisms do consume ammonia nitrogen they form neither nitrites nor nitrates and ultimately yield up the nitrogen which they have absorbed in the form of albuminoid nitrogen. Some investigators have laid stress upon the consumption of organic matter by mold-fungi (like *Leptomitus* and *Beggiatoa*), by Protozoa, worms, insects, Crustacea, mollusks and fishes. These organisms, too, while they temporarily alter the condition of their food material, do not render it on the whole any more stable or less putrescible. The only forms which so far as we know are able to effect extensive simplifications and oxidations of organic matter are certain bacteria. It is most probable that the active agents in the chemical self-purification of streams are microbes of the same types as those which in sewage filters

change relatively large amounts of organic matter, more or less quantitatively, into the mineral form.

There is at any rate no doubt that the process of self-purification, so far as the organic matter is concerned, — by whatever agents it may be carried out, — is essentially a process of oxidation, in the course of which substances of the ammonia nitrogen and albuminoid nitrogen classes are changed to nitrites and eventually to nitrates.

Dr. Adeney (Letts and Adeney, 1908) in a series of elaborate and beautiful experiments has shown that the process of self-purification may be divided into two phases. In the first stage carbonic acid, water, ammonia and humus like organic compounds are formed. The oxidation of the carbon atoms in the molecule is the particular characteristic of this phase. In the second stage the humus matters and the ammonium compounds are further changed, with the final production of nitric acid, as well as carbonic acid and water. This is pre-eminently the phase of nitrogen oxidation. The final change may be very slow, and the relations between the substances involved indicate that it is highly complex, the ammonium compounds not being readily oxidized in the absence of the humus materials.

Consumption of Oxygen by Polluted Waters. The most delicate measure of the process of chemical self-purification is the change in oxygen content. As self-purification actively proceeds oxygen is rapidly used up, and as the process slackens it is reabsorbed and gradually returns to a normal value. The whole history of the pollution and self-purification of streams may be traced by the diminution and gradual restoration of this constituent. Dibdin's studies of the Thames below London are most significant in this respect and illustrate on a practical scale the enormous volumes of the oxidizing agent needed. He estimates (Dibdin, 1904) that 2000 tons of oxygen are absorbed by the river between Teddington and Southend in this process. The proportion of dissolved oxygen expressed as "per cent of saturation," at various points along the river on the high tide, is plotted in the diagram on page 41 by Winslow and Phelps (1906) from figures given by Dibdin (1904) for 1893–94. As the river enters the city between Kew and Battersea its oxygen content falls from 70 per cent to 43 per cent, and the progressive pollution continues until at Woolwich the oxygen value is only one-fifth

that of saturation. Below Barking Creek the heavy pollution ceases, absorption of oxygen overbalances its consumption, and the normal conditions are gradually restored. The ratio of oxygen to nitrogen, which changes from 1 2 at Kingston, above London, to 1: 62 at Greenwich, is most significant. The same

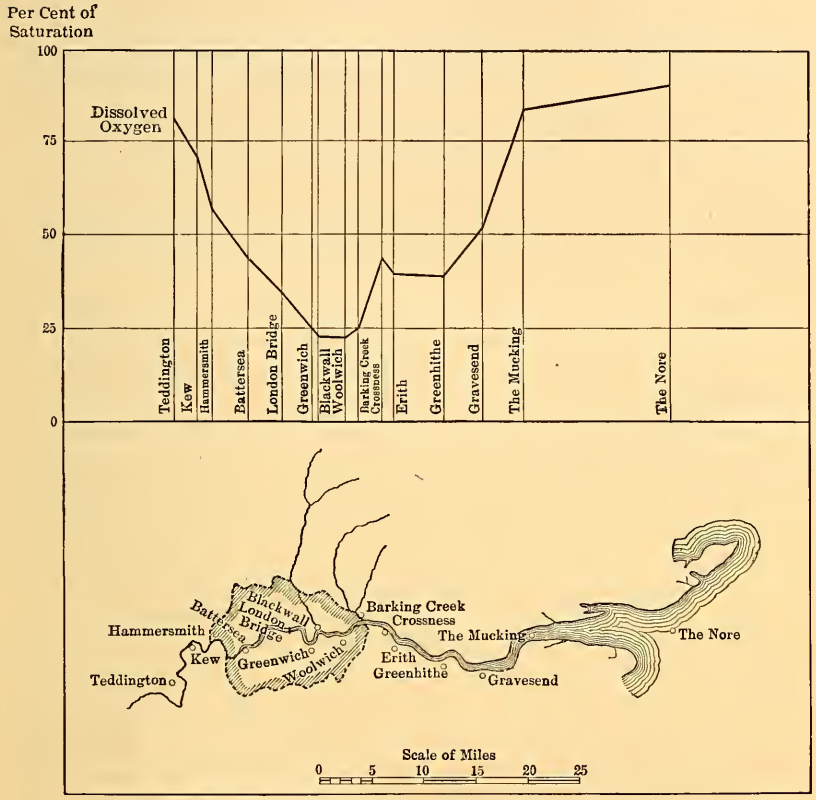

FIG. 5. Dissolved Oxygen Changes in the Thames at London.

general relations are shown in Table XII, quoted by the Connecticut State Sewage Commission (1899).

Sometimes the progressive change in oxygen on the one hand and in oxidizable organic matter on the other hand may accurately record even very slight changes in the physical conditions of the process. A good example of this is the diagram reproduced on page 42 from data collected by Woodman, Winslow and

TABLE XII

DISSOLVED GASES IN THE THAMES ABOVE AND BELOW LONDON, ENGLAND (CONNECTICUT, 1899)

Analyses by Roscoe and Schorlemmer. (Cubic centimeters per liter.)

	Kingston.	Hammersmith.	Somerset House.	Greenwich.	Woolwich.	Erith.
Total volume of gas	52.7	.....	62.9	71.25	63.05	74.3
Carbon dioxide	30.3	.....	45.2	55.6	48.30	57.0
Oxygen	7.4	4.1	1.5	0.25	0.25	1.8
Nitrogen	15	15.1	16.2	15.4	14.5	15.5
Ratio of oxygen to nitrogen	1 : 2	1 : 3.7	1 : 10.8	1 : 62	1 : 5.8	1 : 8.6

Hansen (1902). The Sudbury River is heavily polluted at Saxonville (marked Point of Pollution on the diagram) by the wastes from a woolen mill. Between the one- and two-mile

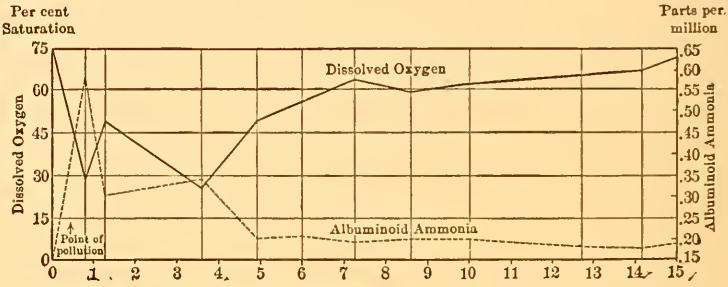

FIG. 6. Pollution and Self-Purification in the Sudbury River.

sampling points a large spring of pure water enters the river bed and causes the dissolved oxygen to rise. From this point for three miles the flow of the stream is rapid and self-purification slow. The river then enters an area of meadows, where it winds along through a weedy channel at a rate of not more than one-fourth mile an hour. Self-purification here goes on actively, so that three miles below the entrance to the meadows and six miles below the mill the chemical constituents of the stream fall to their normal. The striking thing about the diagram is the close inverse relation of the curves for oxygen and albuminoid nitrogen, which illustrates the essential nature of self-purification as a chemical oxidation of putrescible organic matter.

Quantitative Aspects of Oxygen Absorption. Adeney (Letts and Adeney, 1908) made the first careful study of the laws governing the absorption of oxygen from water surfaces and more

recently Black and Phelps (1911) have made important contributions to the subject. They show that in quiet water there are three variables involved — the depth of water, the time and the initial condition of the water in regard to saturation. Winds, tides and the passage of boats which break up the surface are special factors which tend to promote oxygenation.

This study indicated that the importance of reaeration has been greatly overrated and the authors concluded that under most favorable assumptions the absorption of oxygen from the air in New York Harbor will give only an increase of 1.9 per cent of the saturation value in a 24-hour period. Direct aeration of the sewage, to reduce its oxygen requirement by two-thirds before discharge, was recommended in order to ensure a relation between oxygen demand and harbor oxygen sufficient to maintain a final saturation value of 70 per cent in the harbor waters.

Prof. Phelps from a study of the physical constants involved has worked out coefficients for determining the amount of oxygen absorbed by a quiescent body of fresh water of known depth and initial oxygen concentration. The English Royal Commission on Sewage Disposal, in its Eighth Report (R.S.C., 1912) has used these figures in calculating the table below, which shows the dilution necessary under specified conditions of stream flow and sewage and stream composition.

TABLE XIII

DILUTION OF SEWAGE WITH CLEAN RIVER WATER * NECESSARY TO PREVENT DEOXYGENATION BELOW 4 C.C. PER LITER

(R. S. C., 1912.)

Depth of water (feet).	Time taken for river water to mix, (hours).	Rate of reaeration per hour, (per cent saturation).†	Dilution required for		
			Very good filter effluent, (taking up 1.0 p.p.m. dissolved oxygen in 24 hours).	Average sewage taking up 130 p.p.m. dissolved oxygen in 24 hrs.	Strong sewage taking up 200 p.p.m. dissolved oxygen in 24 hours.
2.5	1‡	0.99	0.4	55	85
	6§	0.45	0.9	120	185
5.0	1‡	0.33	1.2	160	250
	6§	0.15	2.5	360	550
10.0	1‡	0.12	3.0	450	700
	6§	0.07	6	800	1200

* Water which will not absorb more than 0.2 c.c. of oxygen per liter in 5 days.

† Saturation at 65° F. is taken as 6.63 c.c. of oxygen per liter.

‡ Arbitrary value assumed for moderately rapid stream.

§ Arbitrary value assumed for very sluggish stream.

The dilutions obtained in this calculation it will be noted are based on strong English sewage and are much higher than those which have been found effective under American conditions. We cite them here because they indicate the relatively considerable influences of depth and mixture upon the absorption process.

According to this computation the water of a moderately rapid stream 2.5 feet deep will be thoroughly mixed in about an hour and will take up 0.99 per cent of the oxygen necessary to saturate it in this time. Such a stream can digest 1 volume of a very good filter effluent to 0.4 volume of water and 1 part of strong sewage to 85 volumes of water. On the other hand a very sluggish stream 10 feet deep will take 6 hours for complete mixture and in one hour will take up only 0.07 per cent of the oxygen necessary for saturation. One volume of very good filter effluent will require 6 volumes of such water and one volume of strong sewage, 1200 volumes of such water, in order to avoid deoxygenation below 4 c.c. per liter.

Bacterial Purification of Streams. The bacterial self-purification of streams is an entirely separate and independent process from that which has just been considered. While the organic matter of the sewage is being digested and mineralized by the water bacteria, the original sewage bacteria are dying out and disappearing. Sedimentation removes them from the flowing liquid, with the solid particles to which so many of them are attached. Light, predatory micro-organisms of the stream, excess or deficiency of oxygen, osmotic conditions to which they are unadapted, and above all the lack of the rich food supply to which they are accustomed — all these conditions are inimical to the sewage forms.

It is difficult to estimate the exact relative importance of these various factors in self-purification. Taken together they may be grouped under the general term, the environment. River water is an unfavorable environment for many bacteria; and foreign bacteria introduced in sewage, microbes whose home is the alimentary canal, are unable to adapt themselves to it, and die. The process of elimination is gradual, since different species of bacteria and even different individuals of the same species are differently affected. The reduction is greater, however, the longer the process goes on, and for this reason time is the most

important of all conditions for bacterial purification. It used to be said that "running water purifies itself"; but this is just the reverse of the truth. It is stagnant water which purifies itself, for storage is the controlling factor in self-purification.

Self-Purification in the Des Plaines and Illinois Rivers. The most careful study of self-purification which has ever been conducted on a large scale was carried out in connection with the discharge of the sewage of Chicago into the Illinois River; and the results obtained in this investigation serve well to illustrate the general principles involved. The facts of the case in outline were as follows: The sewage of the city of Chicago prior to 1900 was mainly discharged into the south branch of the Chicago River, a sluggish stream flowing east into Lake Michigan. In order to prevent gross pollution of the lake which supplies water to the city, pumps were installed thirty years ago to pump the water of the south branch across a narrow divide through the Illinois and Michigan canal into the Des Plaines River, flowing to the southwest. At times of heavy rain the pumps failed to cope with the natural current of the stream. A new canal was therefore constructed at a cost of nearly forty million dollars to effect a permanent connection between Lake Michigan and the Chicago River on the one hand and the Des Plaines River on the other. This drainage canal is designed to carry a flow of 600,000 cubic feet per minute from Lake Michigan plus the sewage in the Chicago River and the Illinois and Michigan canal. The general relations of the drainage area are indicated in Fig. 7. The Des Plaines enters the Illinois River below Joliet and the Illinois is later further diluted by the Kankakee, the Fox, the Big Vermilion and the Sangamon rivers. At Grafton the Illinois enters the Mississippi. A short distance below, the water of the Mississippi is used as a source of water supply by the city of St. Louis; and on the day the canal was opened, Jan. 17, 1900, the State of Missouri instituted proceedings before the Supreme Court of the United States, praying for an injunction against the State of Illinois and the sanitary district of Chicago. The leading sanitary experts of the country testified in the case, and the complete records, which occupy 8000 printed pages, have been digested and published in brief form by the U. S. Geological Survey (Leighton, 1907).

Fig. 7. Chicago Drainage Canal and Illinois River.

The case was an ideal one for the study of self-purification. The sewage of a large city, carrying four million pounds of urine and fecal matter per day, was discharged into a stream which gave at first a dilution of about one part sewage in ten parts of water. This polluted stream with successive dilutions from purer rivers flowed for a distance of 357 miles in an average time, variously estimated at eight to eighteen days.

TABLE XIV

SELF-PURIFICATION IN THE DES PLAINES AND ILLINOIS RIVERS

January-June, 1900.

Station.	Parts per million.						
	Period of flow, days.	Chlorine.	Ammonia. nitrogen.	Albuminoid nitrogen.	Nitrites.	Nitrates.	Bacteria per c.c.
Illinois and Michigan..	....	....	....	....	....	....	.....
Canal, Bridgeport.....	....	96.6	8.05	2.05	0.021	0.074	631,000
Illinois and Michigan..	....	....	....	....	....	....	.....
Canal, Lockport.....	1.6	124.5	10.90	2.07	0.013	0.066	1,755,000
Des Plaines R., Joliet..	1.7	41.5	4.22	0.83	0.021	0.086	744,286
Illinois R., Morris.....	2.5	24.5	2.46	0.60	0.075	0.424	445,000
Illinois R., Ottawa.....	3.4	15.3	1.55	0.41	0.197	0.966	116,000
Illinois R., La Salle....	....	17.5	1.05	0.43	0.109	0.979	94,000
Illinois R., Henry.....	5.3	13.3	0.92	0.38	0.102	0.800	64,200
Illinois R., Averyville..	....	13.5	0.81	0.37	0.004	1.150	51,800
Illinois R., Wesley.....	....	12.0	0.56	0.41	0.083	1.030	36,800
Illinois R., Pekin.....	9.9	12.3	0.70	0.43	0.060	0.990	68,400
Illinois R., Havana....	11.4	11.2	0.60	0.36	0.065	0.570	23,100
Illinois R., Beardstown.	12.8	10.7	0.69	0.44	0.106	0.685	28,200
Illinois R., Kampsville.	15.5	11.3	0.66	0.44	0.044	0.870	33,700
Illinois R., Grafton....	17.7	9.8	0.46	0.42	0.031	1.060	21,000

The most important features of the resulting self-purification are indicated in Table XIV. The estimates for the period of flow are taken from the testimony of Isham Randolph, C. E. According to the experts on the St. Louis side these figures should be cut in half. The analytical data are from the testimony of Prof. E. O. Jordan.

The reduction of chlorine, plotted in the diagram (Fig. 8), measures pretty closely the amount of self-purification by dilution alone. From the Illinois and Michigan canal at Lockport to the Des Plaines River at Joliet the chlorine content falls to one-third its former value, as a result of the dilution furnished by the drainage canal. The further progress of the curve shows a

fairly steady decrease, the final value reached being about eight per cent of the value at Lockport.

The changes in organic matter during the same period of flow are indicated by the curve for ammonia nitrogen in Fig. 8. The decrease of ammonia nitrogen parallels pretty closely the decrease in chlorine; but it is more marked. The final ammonia

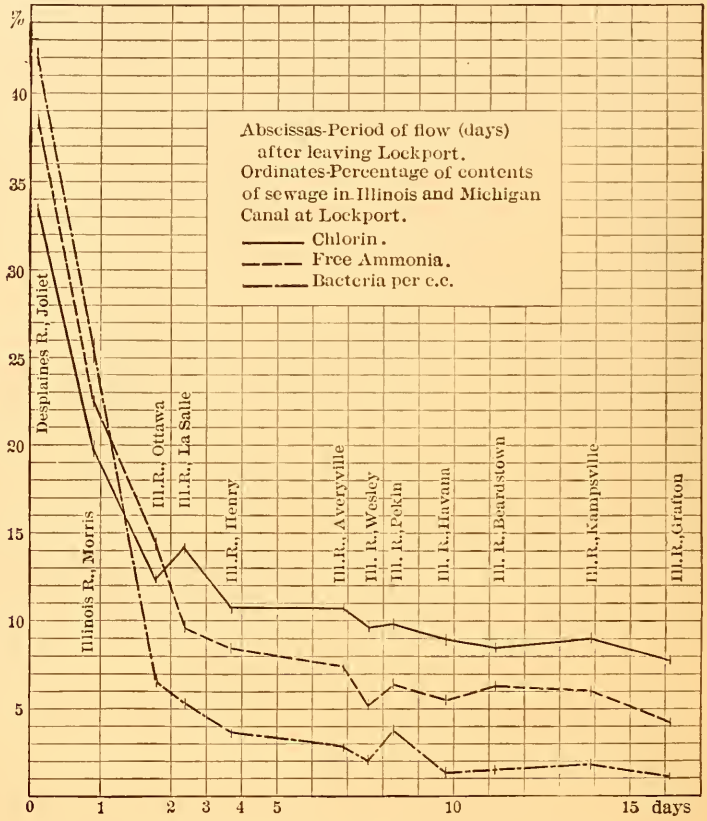

FIG. 8. Self-Purification in the Des Plaines and Illinois Rivers.

nitrogen value at Grafton is only four per cent of the value at Lockport. Here, as in the case of chlorine, it is probable that dilution is responsible for most of the improvement; but some of the ammonia nitrogen not affected by dilution has been removed by oxidative processes. The nitrite and nitrate figures show the progress of the changes which are at work. Where the

process is most active, from Ottawa to Henry, nitrites rise to a maximum. From Ottawa to Wesley nitrates are increasing. At Pekin, the discharge of sewage from Peoria causes some temporary reducing changes, followed again by an increase of nitrates at Kampsville and Grafton.

The bacterial self-purification which accompanies these chemical changes is also indicated in Fig. 8. A steady decrease is apparent from Lockport to Wesley. At Pekin there is a notable increase, due to the sewers of Peoria, followed by a second self-purification. Comparison of the total improvement effected

Fig. 9. View of the Chicago Drainage Canal at Willow Springs (Courtesy of Robert R. McCormick).

shows a bacterial reduction of nearly ninety-nine per cent from Lockport to Grafton. The reduction here is considerably greater than in the case of the organic constituents. Aside from dilution, true self-purification has played an important part.

It is interesting to notice, as epidemiological evidence of the value of self-purification, that no striking excess of typhoid fever at St. Louis followed the opening of the Chicago drainage canal. The experts for the plaintiff were able to show a slight increase in typhoid fever; but this might well have been due to the pollution of the Mississippi River from sources other than Chicago, or to causes other than water. The Supreme Court

finally dismissed the case without prejudice, on the ground that damage to St. Louis was not proved.

Subsequent History of the Chicago Drainage Canal. The success of Chicago in its litigation with the State of Missouri was, however, by no means the last chapter in this episode. As the sewage discharge into the Drainage Canal increased serious complaints arose nearer home than at St. Louis, as to the condition of the waters of the canal itself and those of the upper Des Plaines river. During the summer the oxygen in the canal water was frequently exhausted and fish life was practically eliminated. A report by G. M. Wisner showed that of 21 tests made at Lockport between May 10 and Sept. 28, 1911, 12 showed no dissolved oxygen while the other 9 showed 6 per cent or less, while considerable sludge deposits had accumulated in sections of sluggish flow.

In 1914 Messrs. Alvord, Walker and Pierce made a report to the Chicago Real Estate Board, in which it was pointed out that while the United States had only authorized the Sanitary District to withdraw 4167 cubic feet per second of diluting water from Lake Michigan, the dilution factor called for by the state law creating the Sanitary District (20,000 cubic feet per minute per 100,000 population — 1 part of sewage to 10–20 parts of water), would require nearly 8000 cubic feet per second of diluting water to care for the sewage being discharged at that time. It was furthermore stated that the Sanitary District had as a matter of fact been withdrawing between 7000 and 8000 cubic feet per second, in violation of the Government restrictions, and that "not even this amount of flow is sufficient to effectively purify the sewage actually reaching the main channel in summer, while the sewage from important areas, such as the stockyards, cannot at such seasons be brought into contact with the flow of the main channel until putrefaction is completed, to the great and general nuisance of that vicinity." As a result of these conditions the Sanitary District brought suit against the Federal Government in the hope of obtaining authorization to divert 10,000 cubic feet of lake water per second. It was felt however that even this amount of water would only suffice to stave off trouble for a brief period. The Chicago Real Estate Board therefore called in G. A. Soper, A. J. Martin and J. D. Watson to consider the whole problem. This Commission described conditions in the Chicago River itself as follows:

" Where the currents are not sufficiently rapid to keep the solids moving, deposits take place and these ferment, giving off gases which rise in bubbles to the surface. In disengaging themselves from the material in which they are formed, the bubbles cause eruptions in the deposits, with the result that the black sewage mud is raised and becomes diffused through the overlying water. The evolution of gas, and consequent blackening and production of offensive odors, vary greatly. In the least noticeable cases, the bubbling is scarcely perceptible; in the worse, the eruptions are several feet in diameter and the air smells badly for a distance of several hundred feet. Where fermentation is proceeding actively, the water is in a state of ebullition, the surface looking much as it does at times of rain. The water is then black throughout. In some of the most stagnant places, a scum forms upon the surface of a thickness and consistency which is sufficient to permit small domestic animals to walk upon it. All stages and degrees of fermentation exist in Chicago's waterways."

Of the Des Plaines and Illinois Rivers, below the entrance of the Drainage Canal, they say " there flows for one hundred miles, through the Illinois Valley a discolored, unwholesome and offensive stream.

" For over one hundred miles from Chicago, the inhabitants of the valley seem to have relinquished the most valuable rights of riparian owners The water is not fit to drink, nor to wash in, nor to water stock in, nor for the many other domestic and industrial uses of a normal river. Fish die in it; the thought of swimming in it is repugnant to the senses; boating, far from being a pleasant and healthful diversion, can be enjoyed only by the hardy. The stream flows with the majestic sweep of all great rivers and the banks are overhung with rich, luxurious foliage; but the water is discolored, malodorous, poisonous. Fine black organic sewage mud covers the bottom and deposits on the shores when the river overflows its banks."

The Commission points out that the minimum standards set by the British Royal Commission on Sewage Disposal in its Eighth Report, which require a dilution of 500 volumes for crude or screened sewage, 300 volumes for settled sewage and 150 volumes for chemically precipitated sewage would call, when dilute American sewages are in question, for 15,000, 9000 and

4500 gallons of diluting water per capita per day, while the actual Chicago allowance is only 2338 gallons. The Commission therefore recommended that intercepting sewers should be built and suitable disposal works provided to remove the excessive load upon the diluting powers of the canal and the rivers below, the removal of suspended solids being probably all the treatment immediately required. (Soper, Watson and Martin, 1915.)

The Critical Point in Stream Purification. The experience of Chicago well illustrates both the possibilities and the limitations of self-purification.

The first fundamental which conditions success or failure is of course the presence of an adequate supply of oxygen. There was sufficient oxygen present for the amount of sewage discharged into the Drainage Canal in 1900. There was not sufficient for the amount discharged in 1911.

The actual quantity of oxygen needed for purification is by no means inconsiderable. Dibdin (1903), as shown in the table below, estimates it at from one to three times the weight of the organic substance to be acted upon, for complete oxidation to the mineral form.

TABLE XV

PARTS OF OXYGEN REQUIRED TO OXIDIZE ONE PART OF VARIOUS ORGANIC SUBSTANCES (DIBDIN, 1903.)

Substance.	Oxygen required.				Oxygen already present.	Difference, or additional oxygen required for complete oxidation.
	By the nitrogen.	By the hydrogen.	By the carbon.	Total.		
Gelatin	0.523	0.528	1.333	2.384	0.251	2.133
Chondrin	0.411	0.568	1.310	2.289	0.294	1.995
Albumin	0.457	0.568	1.414	2.439	0.220	2.219
Cellulose, woody fiber	.....	0.496	1.184	1.680	0.494	1.186
Starch	....	0.496	1.184	1.680	0.494	1.186
Fat, stearic acid	....	1.016	2.025	3.041	0.113	2.928

When a gradually increasing amount of sewage is added to a stream the organic matter will be cared for as long as oxygen is present in the water, either free or in easily reducible forms. Additional pollution only makes the process somewhat slower. At a certain point, however, if the increase of pollution continues, the oxidations which are set up will consume all the available oxygen more rapidly than it can be renewed by diffusion

from the surface or in other ways. This is the critical point. Once passed, the whole process changes. Instead of oxidations, a new series of bacterial changes are set up — reducing actions which partially break down the organic compounds into less complex but still unoxidized bodies. This process leads to no final condition of stability, but to a progressive accumulation of half-decomposed organic matter; and it is accompanied by the production of foul-smelling gases, hydrogen sulphide, amines, mercaptans, etc. Putrefaction has taken the place of purification, and a stream has been converted into a septic tank or open cesspool.

Classic Examples of Stream Pollution. The critical point in stream pollution may be passed at times with startling suddenness. Fuller (1912) cites for instance the case of the Passaic River. In 1895 Jersey City still derived its water supply from the river at Belleville. In the next year the Governor appointed a commission to consider means for correcting the nuisances in this entire valley, the river having then become an "open sewer."

Another famous example is that of the Seine below Paris, before the purification of the sewage of the city. The river below the entrance of the main sewers is described as black and stinking. For a long distance it was in active fermentation, and in some places gas bubbles were formed, nearly three feet in diameter (Boudet, 1876). Scum covered the surface at times for a distance of two miles below the sewer outlets. Sludge banks ten feet deep were formed at points of sluggish flow. The analyses on page 54 show the progressive changes in the character of the water.

At a short distance above the main city the pollution began, and continued on a small scale, from minor drains and sewers, to the Pont d'Asnières. Below this point one of the main interceptors and below Pont de Saint-Denis a second, the Collecteur du Nord, were discharged. These successive additions of organic matter reduced the oxygen present to one-ninth of its original amount. For a distance of over ten miles, from Epinay to Écluses de Bougival, the restoration of oxygen was only slightly in excess of its consumption. After this point, however, an improvement began, accelerated by the entrance of the purer River Oise below Pont de Maisons.

TABLE XVI

DISSOLVED OXYGEN IN THE SEINE AT PARIS

(August–October, 1874. Boudet, 1876.)

Station.	Distance below Paris, miles.	Dissolved oxygen, c.c. per liter.
Point d'Ivry	4*	9.50
Point de la Tournelle	0	8.05
Viaduc d'Auteuil	5	5.99
Pont de Billancourt	6	5.69
Pont de Sèvres	7.5	5.40
Barrage de Suresnes	10.5	5.32
Pont d'Asnières	14.5	5.34
Pont de Clichy	15.5	4.60
Pont de Saint-Ouen	16	4.07
Pont de Saint-Denis	17.5	2.65
La Briche	19	1.02
Epinay	19.5	1.05
Pont d'Argenteuil	20.5	1.45
Barrage de Bezous	25	1.54
Ponts de Chaton	28	1.61
Écluses de Bougival	30.5	1.91
Ponts de Maisons	36	3.74
Pont de Poissy	49	6.12
Pont de Triel	53	7.07
Pont de Meulan	58	8.17
Mantes	68	8.96
Vernon	94	10.40

* Above Paris.

An interesting change in fauna and flora accompanies the gradual purification of a stream and the approximate completion of the task is made obvious, even to the eye, by the characteristics of plant growth. At points of extreme pollution no green growth appears, but only the molds and other colorless plants which require organic matter for their food. A grayish or blackish slimy mass of *Leptomitus* covers the rocks. Masses of *Beggiatoa* may float on the surface, and microscopic examination shows only a few Diatoms and Protozoa with occasional filaments of Blue-green Algæ. As purification progresses the mold-fungi disappear and the true green Algæ take their place, thriving on the nitrates formed from the decomposition of organic compounds. *Spirogyra* and *Conferva* and the desmids produce abundant rich green growths. Diatoms are present in greater variety and the *Mastigophera* take the place of ciliated Protozoa.

Practical Limits of Purifying Capacity. It is obviously of great importance to determine how much sewage a given stream

will consume without danger of passing the critical point. This problem is a fairly complex one, since the capacity of a stream at any point must vary with its composition and the composition of the sewage, as well as with their relative flow. Calculations of the British Royal Sewage commission have already been cited in Table XIII. Rideal (1906) has attempted to express the relation between the various factors involved in the form of an equation, $XO = C(M - N)S$, where X = flow of a stream, O = parts of dissolved oxygen in the water of the stream per unit flow; S = volume of sewage or effluent; M = parts of oxygen consumed by a unit volume of sewage; N = parts of available oxygen in the form of nitrites and nitrates, and C = a constant.

The permissible dilution will naturally vary with the degree of oxygen exhaustion accepted as a safe standard. On this point there is at present no very close agreement. Black and Phelps (1911) place the allowable limit at 70 per cent saturation, while Fuller (1912) would set it as low as 30 per cent. Wisner (1911) recommends a minimum of 2.5 parts per million of dissolved oxygen at all times and suggests 3.5–4.0 parts as desirable. Since water when saturated at 10° C. holds 11.31 p.p.m., at 20° C. 9.19 p.p.m., and at 30° C. 7.60 p.p.m. Wisner's minimum corresponds fairly closely to Fuller's standard.

In the first edition of the present book we suggested the intermediate value of 50 per cent saturation. We still feel that this position was a correct one. A saturation of 70 per cent is frequently impracticable, and while a standard of 30 per cent would be satisfactory if maintained "at all places and at all times," it is impossible to examine samples at all places and all times, and experience shows that 50 per cent saturation as an analytical standard does not allow an unreasonable margin of safety.

Certain rules of a simple character may be laid down for practical purposes as a result of numerous studies of various rivers, and on the assumption of a fair normal composition for stream and sewage. Stearns (1890) gave as his opinion that if the flow of a stream is less than 2 cu. ft. per second for each one thousand persons connected with the sewers flowing into the stream, the amount of pollution is inadmissible. If the flow is greater than 8 cu. ft. per second for each one thousand persons connected

with the sewers, the amount of pollution in the stream will not cause offence. Between the two limits is debatable ground. Rudolph Hering (1888), drawing his conclusions from the work of the Massachusetts State Board of Health, states that if the flow is less than $2\frac{1}{2}$ cu. ft. per second per thousand persons (or one gallon per minute per person) an offense is almost sure to arise, but when it exceeds 7 cu. ft. per second per thousand persons, safety is assured. Mr. Goodnough, in a Report to the Committee on the Charles River Dam, 1903, states: "Omitting reference to objections caused by the manner of discharge of sewage and objections which may be due to various other circumstances, and considering only the question as to whether objectionable conditions exist in the various streams into which sewage is discharged by reason of the quantity of sewage discharged, an examination of all the information available from the investigations that have been made shows that where the flow of a stream exceeds 6 cu. ft. per second per 1000 persons discharging sewage, objectionable conditions are unlikely to result."

The statements of Mr. Stearns, Mr. Hering and Mr. Goodnough had regard only to the decomposition of organic matter causing offense and did not contemplate the use of water for manufacturing purposes, or the use of a stream for a water supply; with our present knowledge it cannot of course be stated that any degree of dilution will make the water entirely safe for the latter purpose.

It should also be noted that Mr. Stearns in giving his opinion regarding pollution says: "My conclusions relate to the pollution of the water itself, as if the sewage was emptied into a stream of unvarying volume, flowing with sufficient rapidity to prevent deposits. If, instead, the sewage is turned into a stream where it is ponded by a dam, or if there are ponds on the stream below the point of discharge, the solid particles of the sewage may accumulate and decompose, giving off offensive gases. This is more likely to occur if the deposits are covered with foul water in which the dissolved oxygen has been used up, because the decomposition will then be putrefactive rather than a process of oxidation. The fluctuations in the height of a stream, where they cause large areas to be alternately covered with water and left bare, are also unfavorable for the proper disposal of sewage.

In short, there are many things, such as the variations in the volume flowing in a stream occasioned by its use for mill purposes, the amount and character of manufacturing wastes, and the subsequent use of the water for different kinds of manufacturing, which require careful consideration in each case, and often a considerable variation from any general rules which may be laid down."

A still more useful method of statement involves the relative volume of sewage and stream flow, assuming otherwise average conditions. Johnson (1905) has converted Hering's and Goodnough's figure into dilution volumes as follows:

TABLE XVII

PROPORTIONS OF SEWAGE WHICH CAN BE DISCHARGED INTO A STREAM WITH SAFETY

(Johnson, 1905.)

Authority.	Nuisance probable.	Nuisance improbable.
Hering	1 in 16	1 in 45
Goodnough	1 in 23	1 in 36

These figures correspond in a general way to those derived from a consideration of the amount of oxygen necessary to oxidize the organic matter in an average sewage, assuming no appreciable renewal by reaeration. Thus Metcalf and Eddy (1915) assume that 500 parts of oxygen per million parts of sewage will be needed to oxidize completely the organic matter present. This will amount to 4000 pounds per million gallons. Fresh water at ordinary temperature and pressure contains 10 parts per 1,000,000 of oxygen, or 83 pounds per million gallons. Fifty parts of water to one part of sewage would thus be indicated as a minimum diluting volume.

Roughly, then, it may be said that a stream will purify one-fiftieth of its volume of sewage, but not one-twentieth. With a very sluggish stream sludge banks may accumulate and cause local nuisance with a dilution as high as one hundred parts or more of water to one of sewage (as in the Spree at Berlin). In summer the danger of putrefaction is much greater than at other times. Stream flow is at a minimum, while high temperature makes bacterial decomposition rapid and the need for

oxygen correspondingly immediate. Thus a stream which is in good condition for most of the time may be dangerously near the critical point in the late summer. Table XVIII and Fig. 10 on page 59 (Winslow and Phelps, 1906) illustrate this condition as exemplified in the Merrimac River. The flow of this stream is always above Goodnough's limit of six second-feet, but it approaches it closely in September and October. The coincident high temperature leads to a rapid decrease of dissolved oxygen. The importance of the temperature factor is strikingly shown by the curve for November and December, 1899; with no increase in dilution a fall in temperature, with its consequent slackening of fermentation processes, shows a marked rise in dissolved oxygen. Although at the lowest points the dissolved oxygen averages do not show complete exhaustion, the river is sometimes distinctly offensive during the summer. Theoretically, while any dissolved oxygen remains there should not be putrefaction; practically, any value below 50 per cent of saturation may be taken as a danger signal indicating that malodorous conditions are likely to occur.

TABLE XVIII

SEASONAL CONDITIONS IN MERRIMAC RIVER AT LAWRENCE, MASS.

(Winslow and Phelps, 1906.)

Month.	1899.			1900.		
	Flow per 1000 persons discharging sewage (second-feet).	Temperature, deg. F.	Dissolved oxygen (per cent of saturation).	Flow per 1000 persons discharging sewage (second-feet).	Temperature, deg. F.	Dissolved oxygen (per cent of saturation).
January	42.6	34	96.3	18.2	33	81.6
February	26.4	34	88.1	89.1	34	87.8
March	64.5	34	95.6	87.7	35	.....
April	143.2	....	99.3	100.0	41	99.1
May	51.6	58	84.4	54.1	54	.....
June	16.1	73	71.1	21.4	73	62.1
July	13.4	76	66.6	9.8	77	59.4
August	11.3	74	58.3	10.1	75	43.6
September	10.8	67	57.2	8.2	71	32.5
October	9.7	58	53.7	13.6	62	47.6
November	15.1	40	78.1	31.6	46	91.2
December	15.1	36	84.3	36.6	38	98.0
Average	34.5	53	77.8	39.9	53	70.3

Legal Control of Stream Pollution. It is evident that the discharge of unpurified sewage into the nearest stream may often offer an eminently satisfactory method of disposal as far as the polluting community itself is concerned. In such cases it is the other communities below which suffer; and it is important to consider how responsibility can be brought home to the offending party. The first recourse is naturally to the common law. The principle is well established that every riparian owner has a right to the reasonable use of the water of the stream and has a right to it in a natural state and unpol-

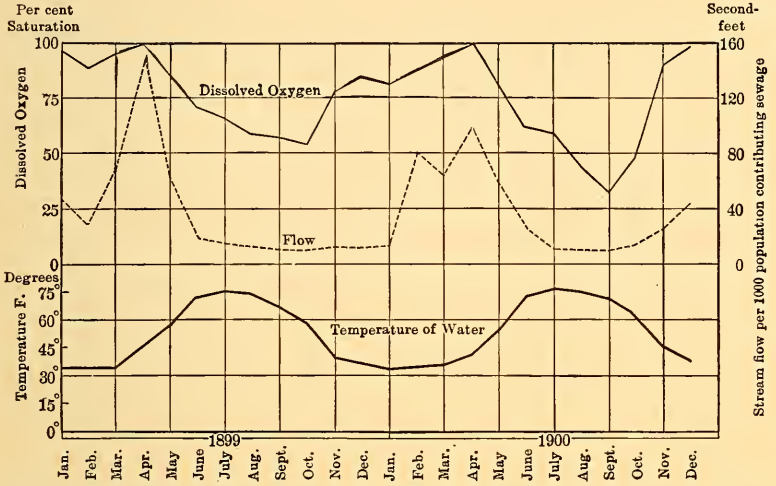

FIG. 10. Relation of Dissolved Oxygen to Flow and Temperature in the Merrimac River.

luted, except for the reasonable use of other riparian owners. The same general law applies to ponds and lakes, unless specifically modified by statute (Choate, 1908). The vital point of this legal principle in practice lies in the definition of "reasonable use." When the common law grew up, reasonable use meant use for washing, watering cattle, irrigating land, bathing, fishing and other purposes incident to agricultural life. All these things are still of course reasonable uses and for whatever detriment they work there is no redress.

With the growth of manufacturing, however, new problems have arisen which are much more complicated. If the farmer

is entitled to use a stream for all his ordinary needs, why has not the manufacturer the same right to use the stream for his needs, which may be primarily the disposal of industrial wastes? Because, as the courts appear to hold, the agricultural use does not seriously affect the character of a stream, while the industrial use may, and often does, affect it very fundamentally. It is true that in Pennsylvania the courts have decided the discharge of water pumped out from the coal mines and laden with acid wastes to be a proper use, in spite of the fact that the water of the stream was rendered practically useless for domestic purposes. The general trend of opinion is, however, in the other direction. The Supreme Court of Massachusetts in the case of Parker against the American Woolen Company rendered a sweeping decision in the following terms: "The plaintiff will restrain the defendant from discharging into the stream any noxious or offensive substances to such an amount or in such a quantity as to affect noticeably or appreciably the purity of the water, when it reaches the plaintiff's premises, so as to render it materially less fit for drinking or for other uses than it was when it entered the defendant's premises."

Whatever the ultimate position of the courts may be in regard to industrial wastes, the principle is well established that the discharge of domestic sewage in such amounts as substantially and appreciably to alter the natural character of a stream is an injury for which both damages and injunctions may be obtained by riparian owners.

The construction of one of the largest sewage disposal plants in the United States, at Columbus, Ohio, was undertaken only under the pressure of litigation, in the course of which the city was directed to pay heavy damages to riparian owners along Alum Creek. The courts of New Jersey have taken an advanced stand on this subject in several recent cases. It has even been affirmed in one instance that surface drainage, when carried in artificial pipes rather than over the surface of the ground, became by that fact polluting material and might not be discharged into a watercourse unpurified.

Experience in the United States with Dilution of Sewage in Lakes and Streams. Aside from the Chicago experience, which has been discussed above, the most important studies of the process of self-purification are perhaps those which have been

carried out by the Massachusetts State Board of Health in 1890, in 1902 and in 1908; and by the Ohio State Board of Health in 1897. Extensive investigations of this point have also been conducted at Rochester, N. Y. (Fisher, 1913), Toronto, Canada, Cleveland, Ohio, and Milwaukee, Wis. The first three cities have decided to utilize dilution in the Great Lakes as a means of disposing of the sewage after a greater or less degree of clarification. At Milwaukee, however, sewage studies were undertaken with a view to demonstrating some practical means of purifying the sewage to a high degree from both physical and bacterial standpoints, in order to protect the city's water supply and avoid the necessity for water filtration. Toronto installed Dortmund tanks, and Rochester, Imhoff tanks, while the type of construction at both Cleveland and Milwaukee is still under discussion.

One of the most successful examples of disposal by dilution is that of Washington, D. C. This project was recommended by Hering, Stearns and Gray in 1889. The sewage of over 300,000 people is discharged by two outlets about 700 feet from shore and 28 feet below the surface. Colonel W. M. Black reported after an inspection in 1909 that the exact point of discharge could scarcely be located from the surface. There was an area of discoloration about 30 feet wide by 250 feet long and only a very slight odor was perceptible.

Disposal of Sewage in Tidal Waters. Cities situated on the seaboard are fortunate in having the most favorable conditions for disposal by dilution. The volume of diluting water is large and the pollution of water supplies is not to be feared. On the other hand sea water normally contains about 20 per cent less oxygen than fresh water, other things being equal; and salt water exerts a specific precipitating action upon sewage solids so that there is a greater tendency to form sludge banks in marine disposal.

In the case of New York City the question of harbor pollution has been carefully studied by several Commissions (Soper, 1906). Some 450,000,000 gallons of sewage are daily discharged into the harbor. The water shows evidences of pollution (0.1 part per million of ammonia nitrogen), and analyses are not much better on the incoming than on the outgoing tide, showing that much of the polluting material is washed back and forth instead of being

flushed out to sea. The harbor in general takes care of all that is discharged into it, without creating any general nuisance. At particular points, however, conditions are most objectionable as in Newtown Creek, a tributary of the East River, and in the Gowanus Canal in Brooklyn; and the Metropolitan Sewerage Commission in its last report states that the harbor in general "is more polluted than considerations of public health and welfare should allow."

The Metropolitan Sewerage Commission estimated in its 1910 Report (N. Y., 1910) that the dilution of sewage in the harbor at that time was 1 in 32 (4.7 cu. ft. of diluting water per second per 1000 population) to be reduced to 1 in 13 (2.65 cu. ft. per second per 1000 population) in 1940. The plans at present adopted (1918) contemplate disposal by dilution from some thirty outlets after preliminary removal of the coarser suspended solids.

Local nuisances have led in many seaport towns to the discontinuance of discharge from pierheads and the substitution of intercepting and outfall sewers discharging at some distance out to sea. Boston offers a good example of this latter method. Since 1895 two main sewers have discharged into the harbor, serving the city and surrounding metropolitan district, which includes 25 cities and towns, with a territory of nearly 20 square miles. The sewage of the region north of the Charles flows continuously from an outlet near Deer Island Light and averaged 59,800,000 gallons per day in 1908. The sewage from the region south of the Charles has been discharged since 1884 at Moon Island, nearer the center of the harbor. Here, in order to protect the adjacent shores, it has been thought necessary to hold the sewage in four masonry basins and to discharge it only on the outgoing tide. An average of 87,661,058 gallons a day passed out at the outlet in 1908. On September 19, 1904, a third outlet was opened to take the sewage from certain high-level regions in the south metropolitan district. This discharges continuously in the outer harbor near Nut Island and delivered 37,800,000 gallons per day in 1908. Experience has shown that no serious nuisance is caused by the Deer Island and Nut Island outlets. The sewage at Deer Island disappears within $1\frac{1}{4}$ miles of the outlet, while off Moon Island the sewage stream may be traced outward round the south end of Long Island for perhaps

two miles. In both cases passing boats find the immediate vicinity of the outlet unpleasant, and near Moon Island the value of property on the mainland is said to be affected. No serious menace to health, however, is involved and the sewage apparently produces no permanent damage in the harbor. So popular is this method of disposal in the sea that according to a review made by the Massachusetts State Board of Health in 1902 (Massachusetts, 1903) nearly one-half the population of that State was tributary to such systems. In general they have proved successful, although a serious nuisance has been created in some places, as at Lynn, where the sewage was discharged in shallow water and over tidal flats. It is certain that such methods of disposal will prove less and less satisfactory from year to year as the volume of sewage and the concentration of shore population increase.

Where tidal waters are used for the cultivation or storage of shellfish the problem takes on a new aspect. Direct discharge of unpurified sewage is here undesirable because the disease germs present menace the purity of an important food supply. This question has been exhaustively treated by Fuller (1905*b*), who shows that the annual value of the shellfish taken along the Altantic and Gulf coasts is over fifteen million dollars. Nearly half of this crop is grown in the waters of the Delaware and Chesapeake Bays, into which the sewage of Philadelphia, Wilmington, Baltimore and Washington is discharged. In England, too, this problem has attracted wide attention. The Royal Sewage Commission in an extensive report concluded that the consumption of polluted shellfish was a grave evil, but that it must be met, less by restricting sewage disposal than by regulating the taking and storing of shellfish (R. S. C., 1904*a*). With such an industry as that of Chesapeake Bay this method of procedure will scarcely suffice. When the city of Baltimore undertook a study of its sewage problem the Sewerage Commission required that "the effluent proposed to be discharged into the Chesapeake Bay or its tributaries in the system to be recommended by the engineers shall be of the highest practicable degree of purity." In pursuance of this requirement and as a concession to the demand for a bacterially purified effluent which should not menace oyster beds, the Commission included in its preliminary estimates over a million dollars for supple-

mentary sand filters in addition to the trickling filter plant required for organic stability (Baltimore, 1906).

The development of processes for the disinfection of sewage (see Chapter XIV) and more recently the demonstration that oysters may be purified by storage for a brief period in pure water have made such extreme refinements generally unnecessary.

A very curious problem has arisen at Belfast, Ireland, in connection with the discharge of sewage into tidal waters. The trouble is here due to immense growths of the green sea-lettuce, *Ulva*. This alga thrives not only on ammonia nitrogen, but, like other green plants, on nitrates as well, and complete oxidation would leave the sewage nitrogen still in a form to favor its development. Professor Letts (1908) has devised a special process of purification to meet this need, which is discussed in Chapter X.

The Problem of Dispersal. It cannot be too strongly emphasized that the success of disposal by dilution depends on a prompt and intimate mixture of the sewage with the diluting water and its contained oxygen. Again and again we find serious nuisances produced by discharge at pierheads and in shallow water, where the total volume of diluting water available would be ample if promptly utilized. We have quoted above a clear statement of this problem by Mr. Stearns and examples of failure due to its neglect at New York and at Lynn, Mass., have been cited. Black and Phelps (1911) estimated that if the sewage discharged into New York Harbor could be discharged in the strong current at its two entrances the standard of 70 per cent dissolved oxygen could be attained without any preliminary treatment, while under present conditions of discharge it would be necessary to reduce the oxygen requirements of the sewage to one-third their present value. The project for the discharge of the sewage from the Passaic Valley district of New Jersey into New York Harbor provides for discharge in 40 feet of water and by means of a series of multiple outlets covering an area of 3.5 acres.

The General Field for Disposal by Dilution. It is clear from what has been said that disposal of sewage by discharge into lakes, streams and tidal waters is a real method of purification. Under certain circumstances it is the economical and proper one. Thus the plans for sewage disposal at New Orleans pro-

vide for discharge, after rough preliminary screening, into the Mississippi River, at a point well beneath the surface and out from the shore. There are no shellfish layings and no water supplies below; and the vast volume of the river is amply sufficient to absorb the polluting material.

In Germany, where the rivers are large, if not of Mississippi dimensions, disposal by dilution has been extensively adopted, careful screening being generally a preliminary requisite. Thus at Cologne the sewage is passed through a fine screen, and thence through a small sedimentation basin to the Rhine River. It is evident that the lesson of the Chicago Drainage Canal must be taken to heart and that, where the total digestive capacity is not ample, provision must be made for ultimate treatment; while in most cases the removal of floating and settleable solids will be desirable. With the proviso that complete and satisfactory dispersal is secured, we may accept the conclusion of Metcalf and Eddy (1915) that " with suitable provision for screening and, if necessary, preliminary settling or sedimentation of the sewage, the process is one of the most economical, satisfactory and efficient methods of sewage disposal, and the one in most common use."

CHAPTER III

SCREENING AND STRAINING OF SEWAGE

Objects of Screening. Coarse screens were used originally in connection with sewage pumping plants for the purpose of removing from the sewage such materials as would tend to clog and possibly break the pumps; and screens of this type which will hold back sticks, rags and other large objects are almost always desirable in connection with any disposal plant.

Where raw sewage is applied directly to land or artificial filters, whether the filtering material be fine or coarse, it is almost always economical to screen the sewage, and sometimes to a considerable degree of fineness, in order to prevent rapid clogging of the filtering surface and loss in capacity as well as the consequent production of a poor effluent. Where the sewage first passes into properly baffled tanks, before being applied to filters, screens are less essential, although generally desirable. Where sedimentation tanks are operated on the septic plan, however, it is particularly important to screen the entering sewage in order to remove the lighter materials which mass together and form an undue proportion of scum. In case sewage is to be passed through small orifices, as in treatment on trickling filters, rather careful screening will usually be desirable. Allen (1915), quoting McRae, reports that as a result of the introduction of fine screens between the tanks and trickling filters at Baltimore, whereas "two men, eight hours per day, were always required and sometimes more for cleaning nozzles for between six and nine acres of filter area, with the screen in operation two men are required to spend about two hours a day each for the same number of nozzles."

Finally there is a tendency on the part of certain sanitary engineers to use screens of fine mesh for the complete treatment of sewage to be disposed of by dilution in large bodies of water. It has become clear that in many cases the removal of the grosser floating and settleable solids is all that is necessary in order to make disposal by dilution a success. In many German cities

sewage is discharged into the rivers with no purification other than that effected by fine screens; and the use of this method of disposal has led to the design of mechanical and automatically-cleaned screens of numerous types.

If screens are used either in preparation for filtration or for complete treatment before disposal by dilution it is clear that they come in some measure into competition with tank treatment and that in each individual case the relative cost of sedimentation and fine screening must be carefully considered, with reference to the composition of the particular sewage to be treated and the degree of purification necessary to attain. It should be kept in mind, however, that sedimentation tanks, even with short flow periods, remove a type of suspended matter which is not intercepted by screens.

There is a fairly sharp line to be drawn between coarse screens which retain only the larger floating particles and fine screens designed to remove an appreciable proportion of the finer settleable solids. It may be broadly stated that coarse screens are those which have openings of $\frac{1}{4}$ inch or more, while those with smaller openings may be classed as fine screens.

Coarse screens may again be divided into those which are cleaned by hand and those which are cleaned by special mechanical devices; while there are a number of distinct types of fine screens which will be discussed in a later section.

Coarse Bar Screens or Gratings. The most satisfactory type of screen for use at small plants and under conditions where only the coarsest suspended matter is to be removed, consists of a set of parallel iron bars or rods placed on end and spaced so as to provide openings of the desired width. Wooden screens, similarly designed, have been used. The plane occupied by the screen thus formed should be inclined downstream and should make an angle of about 30 degrees with the vertical (see Fig. 25.)

Bar screens may readily be cleaned by means of hand rakes, the teeth of which fit into the spaces between the bars or rods. If it is necessary to place the screen much below the ground level, it can be cleaned by a long-handled rake; or the screen itself may be lifted to the surface for cleaning by means of pulleys operated by hand or power. In some large plants, as at Boston (see Fig. 11), the bars form the sides of a cage with a

perforated bottom, the whole cage being raised by power to an operating floor where it is cleaned by hand.

The usual open space for the bar type of screens is $\frac{1}{2}$ to 1 inch or more. Data for a number of Massachusetts screening plants, cited in the table on page 83, give a fair indication of New England practice at intermittent sand filter plants, but the screens listed are smaller than those generally used elsewhere in

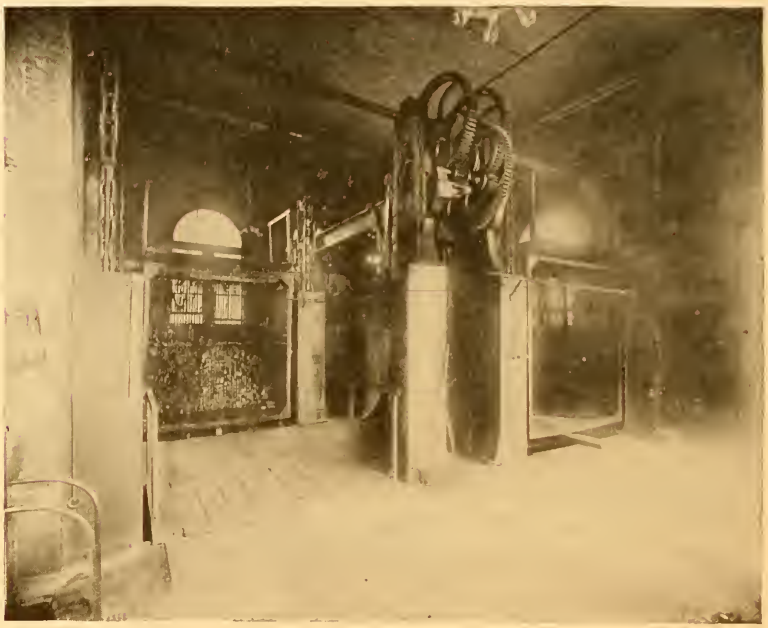

FIG. 11. Cage Screen at Boston, Mass.
(Courtesy of T. F. Bowes.)

the United States. At Columbus, Ohio, there are three sets of screens, the first of 3 by $\frac{3}{4}$-inch bars set 6 inches in the clear, the second of $\frac{3}{4}$-inch rods 1 inch in the clear, and the third of $\frac{3}{8}$-inch rods, $\frac{1}{2}$ inch in the clear. At Washington, Pa., there are two sets of bar screens 4 feet by 3 feet outside dimensions, the first set having an open space of $\frac{3}{8}$ inch and the second set a space of $\frac{1}{4}$ inch. These screens need cleaning every hour or so during the day when the sewage is strongest, but only once in three or four hours during the night. It has not been necessary to em-

ploy any additional attendant to do this work, however, and the advantage derived from using screens with so small an open space is very marked. At the Dorchester plant of the South Metropolitan District of Boston two sets of bars are set one inch apart so that the bars of one set correspond to the openings of the other set. This gives open spaces of about $\frac{1}{8}$ inch in the direction of flow and about one inch diagonally.

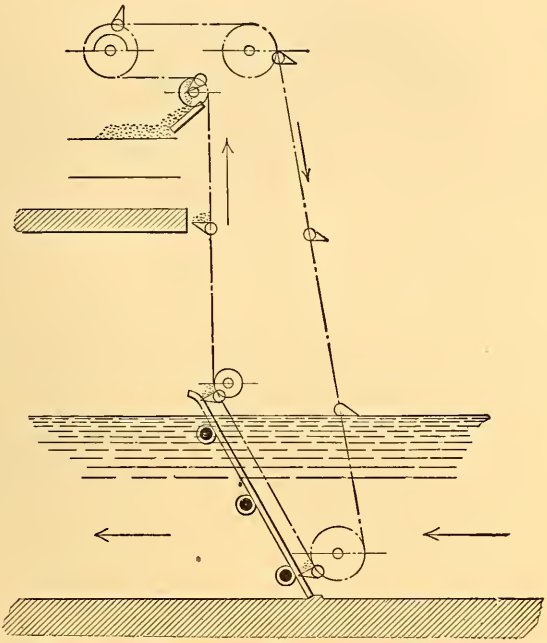

Fig. 12. Stationary Screen with Power-driven Rakes (copied by permission from Dunbar, 1908).

Bar Screens Cleaned by Mechanically Driven Rakes or Brushes. At a number of sewage works, particularly in England, bar screens have been equipped with mechanical cleaning devices to take the place of the hand-operated rake. A common type of this class consists of fixed sets of parallel bars or slats, between which are continuously moving a series of power-driven rakes. The teeth of these rakes are so spaced as to fit exactly the spaces between the slats. The rakes are attached to two endless chains, one at either end of the screen, and by a suitable arrangement of the pulley wheels which guide these chains, the

rakes may be deflected downward on reaching the top of the screen so that the material collected falls onto a platform, from which it is removed, either automatically or by hand (Fig. 12). Raikes (1908) gives an excellent description of a mechanical screen of the bar type manufactured by Messrs. S. S. Stott & Co.

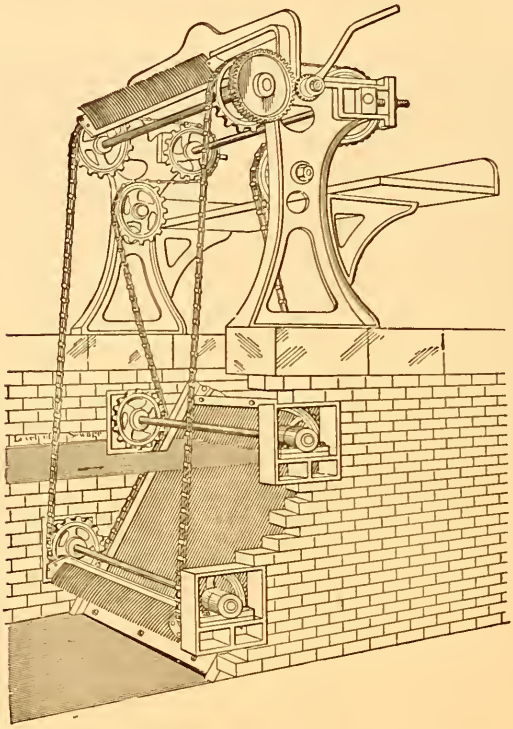

Fig. 13. Sewage Screen Cleaned by Mechanical Raikes (copied by permission from Raikes, 1908).

of Haslingden, which is provided with special appliances for cleaning the rakes (Fig. 13).

There is a famous installation of bar screens cleaned by power-driven rakes at the Clichy pumping station of the Paris sewerage system. A peculiarity of this installation lies in the fact that the bars are inclined upstream. The most important plant on this continent employing the principle of mechanical cleaning of fixed screens is in operation at Toronto, Can.

Mechanically Operated Fine-mesh Screens. With an increasing recognition of the importance of screening out the finer suspended solids from sewage, much thought has been given, particularly in Germany, to the design of fine-mesh screens which would produce a high degree of clarification. Since it would be obviously uneconomical to attempt to clean such screens by hand they are invariably arranged for some form of

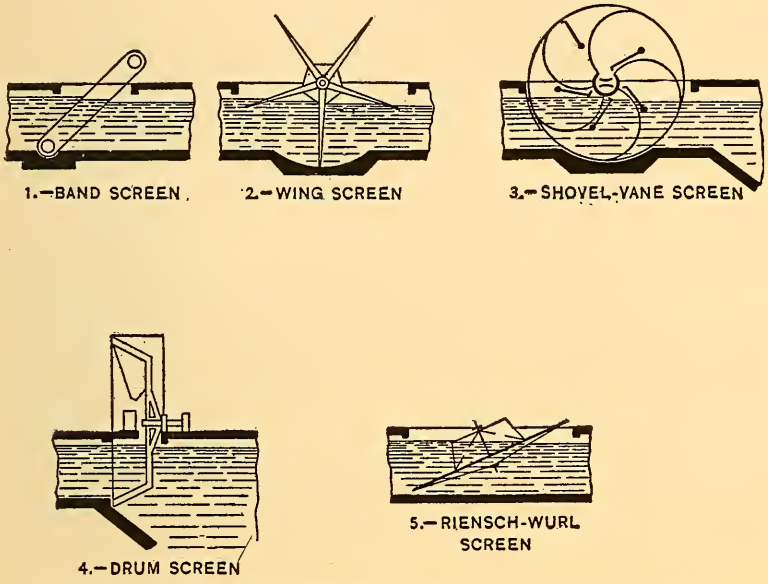

Fig. 14. Types of Mechanically Cleaned Fine Screens (copied by permission from Allen, 1915).

mechanical cleansing, and as a rule this object is attained by making the screens themselves move through the sewage to a point above the surface where the screenings are automatically removed by rakes, scrapers or brushes, by jets of water or of compressed air.

The openings in these mechanically operated screens are sometimes comparable to those of the fixed bar screens (0.4–0.6 inch), but as a rule they are made up of a series of small bars or links, or, in the finer types, of wire mesh or perforated plates. Mr. Kenneth Allen (Allen, 1915) has classified the principal types of mechanical screens which have been introduced under the following five heads (see Fig. 14):

1. The band screen, consisting of a flexible endless band, either of wire mesh or links, which passes over upper and lower rollers.

2. The wing screen, formed of vanes, like those of a paddle wheel, composed of radial bars at uniform distances apart.

3. The shovel-vane screen, similar to the wing screen but with curved vanes.

4. The drum screen, consisting of a cylinder or cone of perforated plates or wire mesh, which rotates on a horizontal axis, the plane of rotation being at right angles with the sewage flow.

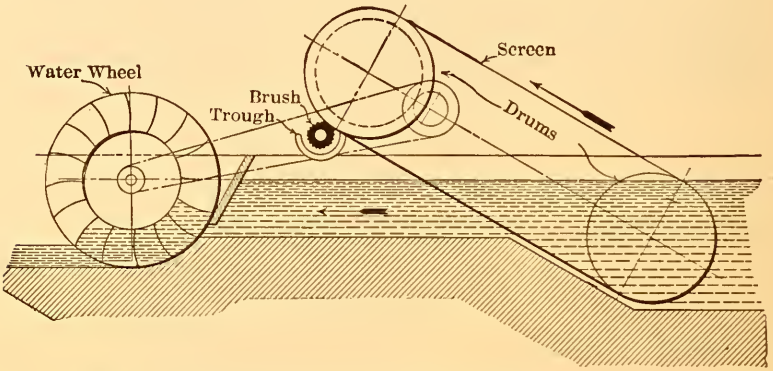

Fig. 15. Revolving Screen, Carshalton (copied by permission from Dunbar, 1908).

5. The Riensch-Wurl screen, which consists of a perforated disc surmounted by a truncated cone, mounted on an inclined shaft.

Band Screens. Band screens have been used at Hamburg, and Göttingen (see Allen, 1915) and quite extensively in England. Fig. 15 represents a screen of this type, manufactured by John Smith & Co., Carshalton, Eng. It consists of a woven-wire screen or sieve, suspended from two revolving cylinders or drums, the lower one being immersed in the sewage transversely across the channel. The lower drum is sufficiently open in construction to allow the liquid to pass through. The intercepted material is caught upon the wire screen, lifted out of the sewage and over the upper drum, where it is removed by a revolving brush and falls into a trough, from which it is taken away, either by hand or by a worm conveyer. The power to

move the screen is generally derived from a water wheel driven by the current of sewage. The screen of this type at Göttingen travels at the rate of $2\frac{1}{2}$ yards per minute. Good results have been obtained with a wire cloth sieve having an area of 1 per cent of that needed for a stationary screen, the linear velocity of the wire cloth being from 0.5 to 20 inches per second, varying with the hourly fluctuations in flow (Kuichling, 1909).

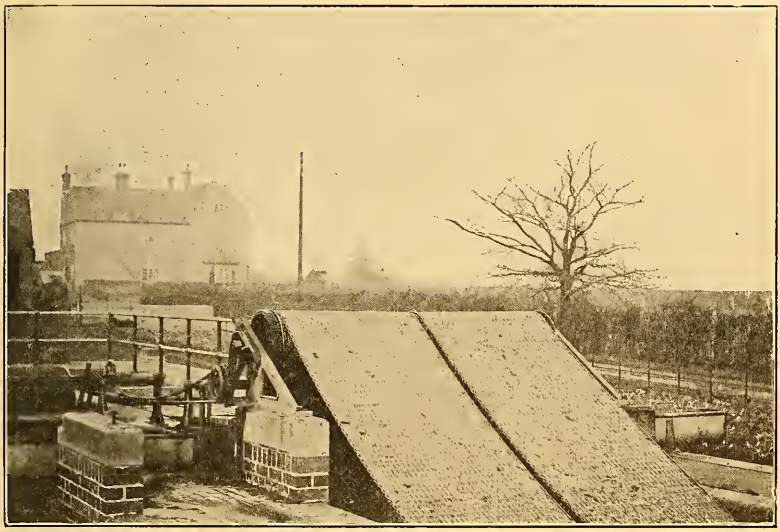

FIG. 16. Revolving Screen at Birmingham, England (courtesy of J. D. Watson).

In the screens of this general design at Hamburg and Glasgow, there are used, instead of woven wire, flat iron bars spaced about $\frac{3}{4}$ inch apart with angle irons so arranged as to prevent the intercepted material from sliding down.

A band screen constructed on the link belt plan in small separate sections of Monel metal wire mesh (40 meshes to the inch) has been used by Jennings for the treatment of Chicago Stock Yards sewage. The screenings are blown off by compressed air and with the concentrated sewage treated (700 parts per million of suspended solids) a removal of 63 per cent is reported (Allen, 1915).

Wing and Shovel-vane Screens. Fig. 17 illustrates the wing screen introduced in 1899 by Schneppendahl at Wiesbaden (Dunbar, 1908). Each of the five or six wings consists of a flat rigid bar or slat screen having an open space of $\frac{1}{4}$ inch or more. The bed of the sewer or chamber beneath the screen is hollowed out in the form of a segment of a circle and the screen is revolved in a direction opposite to the current of sewage. The accumulated screenings are brushed off into a trough as they reach the

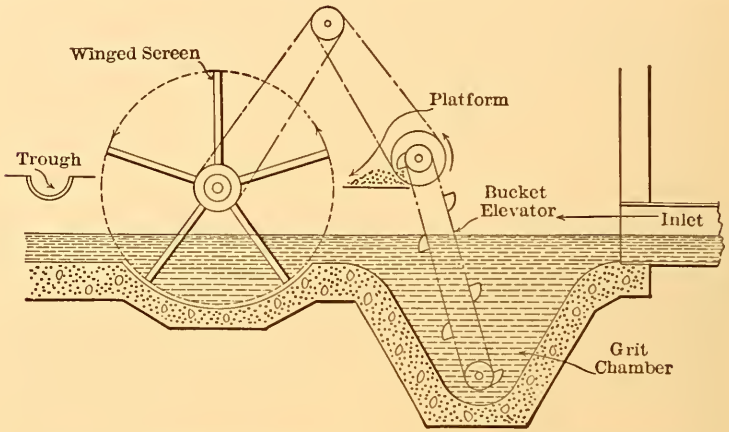

Fig. 17. Automatically Cleaned Grit Chamber and Wing Screen at Wiesbaden.

top. This type of screen is usually preceded by a grit chamber as indicated in the figure.

Allen (1915) describes wing screens in operation at Frankfort and Elberfeld in Germany and at Bradford, England, and shovel-vane screens (see Fig. 14) at Strassburg and Gleiwitz and at Temesvar in Hungary.

Drum Screens. The drum type of screen was first used by Metzger of Bromberg and was later, according to Allen (1915), improved and patented by Windschild. The sewage enters one end of a cylindrical drum which has its curved portion made up of perforated plates. The sewage passes out through these perforations while the screenings are carried upward by the rotation of the drum, being prevented from falling, till they reach a can hopper or conveyer near the top, by a curved guide plate. Clogging materials are removed by an air jet delivered from a

swinging nozzle on the exterior. Allen (1915) describes studies on the operation of these screens at Bromberg, Mainz, Trier, and Osnabrück.

The most interesting revolving screen which has been designed in the United States is a screen of the drum type installed at Reading, Pa., by O. M. Weand (Hering and Fuller, consulting engineers for the plant). This screen, called by the designer a " segregator," consists in its improved form of a cylin-

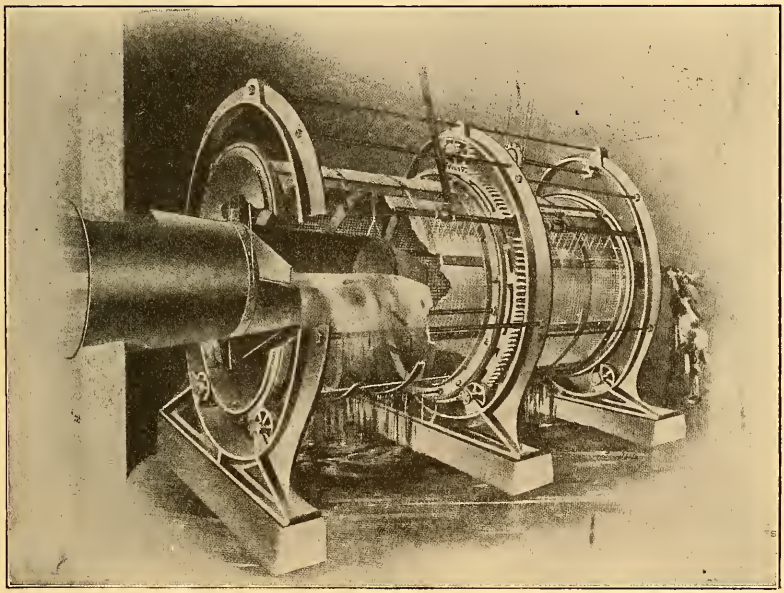

Fig. 18. General View of Reading Screen.

drical iron framework about 6 feet in diameter and 16 feet long. Securely clamped to this framework is a brass wire (No. 12 gage) screen having a $\frac{5}{8}$-inch mesh. This serves as a support to the brass wire cloth, having 40 meshes to the inch, which constitutes the screening surface. The latter is installed in segments which can be readily replaced as needed.

" The apparatus shown in Figs. 18–20 is placed in a pit adjacent to the engine room of the station where the sewage is pumped to the purification works. The sewage from the main sewer is discharged through a 24-inch pipe into the barrel of the screen and passes outward through the wire cloth to the bottom

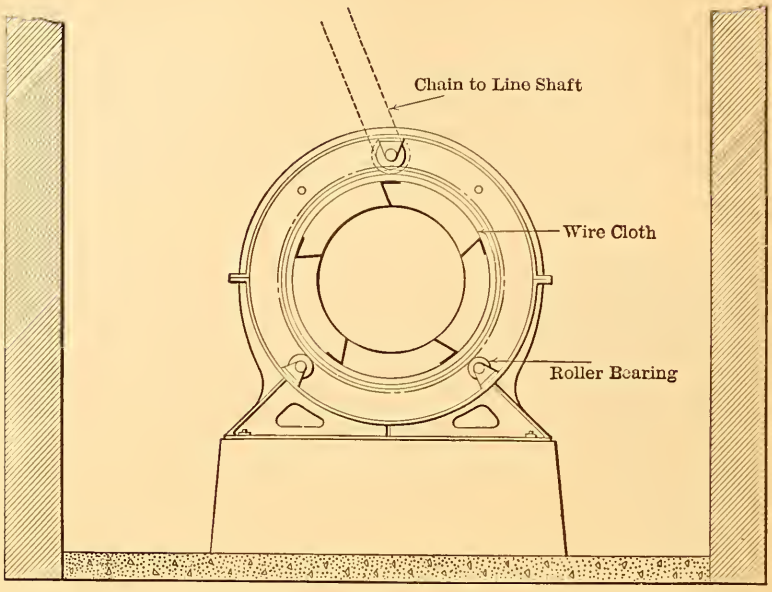

FIG. 19. Section of Reading Screen.

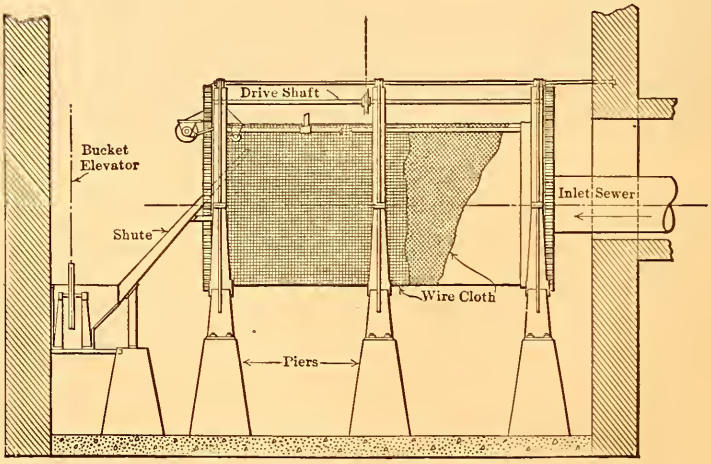

FIG. 20. Elevation of Reading Screen.

of the pit and thence to the pumps. The segregator is revolved continuously at a rate of about six revolutions per minute, by means of power derived from one of the engines, transmitted through a line shaft, chain and sprocket to the driving shaft and thence through spur wheels, engaging gear rings which are rigidly attached to either end of the cylindrical framework.

"The accumulated screenings are removed from the wire cloth by means of jets of water directed against the outside of the screen. The pipe supplying the water has a longitudinal motion which covers the whole screen area by the water from the jets. This washing process causes the screenings to fall to the lowest point of the interior of the cylinder, where they are continuously pushed forward by a worm conveyer to the lower end of the screen. Rotating scoop buckets then deliver the screenings through a shute to a platform in the bottom of the pit. Thence this material is elevated by a belt conveyer to the operating floor above, where it is dried in a centrifugal dryer, mixed with coal and burned under the boilers. The screenings, before drying, range from about 20 to 30 cubic feet per million gallons of sewage and contain about 90 per cent of water, which is reduced to 75 per cent by the centrifugal process."

Weand screens have been in use at Brockton, Mass., Atlanta, Ga., and Baltimore, Md., as well as at Reading. At Reading and Atlanta they have been abandoned. According to Metcalf and Eddy (1916) the Reading screen removed about 20 per cent of the suspended solids in the sewage but they quote E. S. Chase, the engineer in charge of the operation of the plant as follows: "The amount of material actually removed from the sewage by the screen was undoubtedly relatively large, but the cost of its operation, maintenance and especially repairs was such that I advised the discontinuance of its operation." In regard to the Brockton installation the Sewer Commissioner in his report for 1913 states that while the screen "has given quite satisfactory results as regards screenings, its cost of operation and especially its maintenance in good repair have been very costly compared with newer methods of obtaining the same results." The Baltimore drum screens on the other hand have proved very satisfactory.

The Riensch-Wurl Screen. The screen which has been most widely advertised and which is at present attracting most

general notice in the United States is the type first devised by Riensch for use at Dresden and after his death perfected by Wurl of Berlin.

The Riensch-Wurl screen consists of an annular disc surmounted by a truncated cone, in the general shape of a hat with a low conical crown. It rotates on a shaft inclined from 10° to 30° from the vertical and is placed at such a level that a part of the cone and a larger fraction of the disc pass periodically through the flowing sewage. Both cone and disc consist of brass plates,

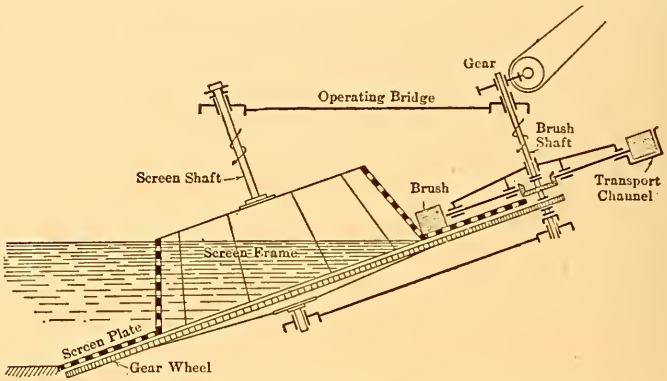

FIG. 21. Elevation of a Riensch-Wurl Screen (copied by permission from Allen, 1915).

perforated with slots 0.03 to 0.20 inches in width and one to two inches long. The screenings which collect upon the plate are removed by a series of hogs' bristle or pissave fiber brushes, which revolve so as to pass across the disc as it emerges from the sewage, while another set of vertically placed brushes cleans the cone. Such perforated plates are believed to be more effective than link screens or meshes in removing straws, fibers, etc.

Riensch-Wurl screens have been used in Europe at Bremen, Christiania, Stettin, Karlsruhe, Mainz, Strassburg, Petrograd, Toulon, Astrachan, and some fifty other places, as well as at Dresden. In this country screens of this type have been installed at Long Beach, Cal., Daytona, Fla., Cleveland, O. (experimental) Wildwood, N. J., Rochester, N. Y., and Brooklyn, N. Y. The Long Beach plant (Rowe, 1916) was the first to be installed, having been in operation since July, 1915. Two

screens have been operated in Brooklyn since August of the same year and have proved very satisfactory.

The operation of the Riensch-Wurl screen under rather severe conditions has been somewhat exhaustively studied by one of us (R. W. P.) at Cleveland. Small stationary experimental mesh screens in preliminary tests had indicated a removal of 25 to 28 per cent of suspended solids with a concentrated day sewage and short periods of operation (Hoffman, Pratt, Hommon, 1914) and the engineers of The Sanitation Corporation claimed

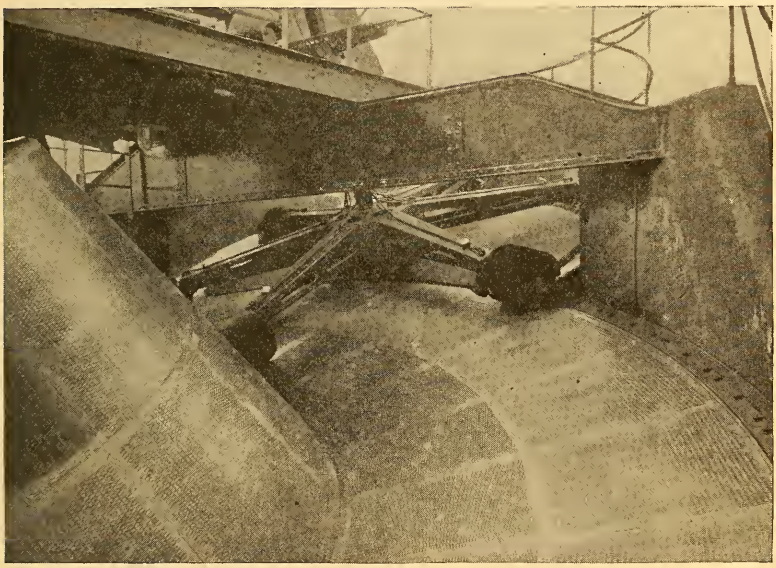

FIG. 22. View of a Section of a Riensch-Wurl Screen in Service Showing the Screen Cone and Disc Plate coming away from the Brush Sphere (courtesy of The Sanitation Corporation).

an efficiency of 50 per cent for the Riensch-Wurl apparatus. The contract for the test installation of Riensch-Wurl screens at Cleveland was a somewhat unusual one in that it provided that 80 per cent of the contract price of the screens was to be paid only after actual tests had demonstrated the fulfilment of the following specifications:

a. Continuous operation during 90 per cent of a 3 months' testing period.

b. The treatment under a maximum rate of sewage flow of $7\frac{1}{2}$ million gallons of sewage a day with a maximum loss of head of two feet.

c. The accomplishment of results comparable to those obtained from a one and a half hour period of sedimentation.

Fig. 23. Interior of Riensch-Wurl Screen House at Dresden (courtesy of The Sanitation Corporation).

The total period of operation extended from Nov. 19, 1915, to Sept. 10, 1916, a period of 297 days. The official test began Mar. 20, 1916, and occupied 175 days, while final conclusions were based on 100 days of actual operation so selected as to represent the best accomplishments of the screen when considered from all viewpoints.

Many difficulties were experienced in the practical operation

of the screen; and the ideal of operation for 90 per cent of a continuous three months' period was never realized. This was in part the result of the fact that the screen was originally of the over-hung type, supported by bearings entirely above the sewage flow. This proved unsatisfactory and a floor bearing was later installed. Bad breakdowns were caused by a block of wood 4 by 6 by $1\frac{1}{2}$ inches thick which slipped through the preliminary bar screens and by the catching of a measuring rod between the end of one of the revolving brushes and the vertical screen plates. It appeared evident from the experience gained in these tests that the continuous employment of a first-class mechanic for repair work and adjustment would be desirable, and that an extra unit would be necessary in order to secure continuous operation.

The clogging of the screen openings by tar, oil and grease proved a serious factor, and it was finally necessary to provide for cleaning the screen plates with cold and hot water from a nozzle delivering under a pressure of 80 pounds per square inch. About one and a half hours of an operator's time in each eight hours' shift was required for this cleaning operation. At times it was necessary to use gasoline or chemicals to remove the grease.

From the standpoint of capacity the screen (which was a 14-foot circular disc) also failed to comply with specifications. During only one week did the maximum rate reach the guaranteed average of $7\frac{1}{2}$ million gallons, ranging for other weeks between 5.5 and 7.2 million gallons for day flow. The average weekly loss of head ranged between 1.1 and 1.7 feet. An increase in loss of head within these limits did not increase capacity to an appreciable extent on account of increased clogging.

In removal efficiency, results were also disappointing. The compact mat which forms on screens treating European sewage failed to appear. The average actual weekly removal for the fourteen weeks of the official test was only 4 per cent of the total suspended solids, varying by weeks from 2 to 5 per cent. This computation is based on a comparison of weighed screenings with the suspended solids in the screened effluent. By the older method of comparing directly suspended solids in screened and unscreened sewage (which is subject to serious error on account of necessary inaccuracies in sampling crude sewage) weekly average efficiencies varied between 0 and 9 per cent.

Allowing for the fact that the weakness of the night sewage flow operated to reduce the average efficiency of the screens, it was considered by those in charge of the tests that the actual performance of the Riensch-Wurl screens with Cleveland sewage might be considered to equal a removal of perhaps 8 per cent of the suspended solids, far below the 40 to 50 per cent removal which may be attained by sedimentation.

The engineers in charge of the Cleveland work estimated the total annual cost of a screening plant at $23,810 and of a tank treatment plant at $28,540. These estimates included disinfection, and are some 30 per cent too low for present conditions (1918). Assuming a removal of 8 per cent and of 45 per cent of suspended solids, the cost per pound of material removed would be 2 cents for screening and 0.4 cent for sedimentation.

Design of Screen Chambers and Required Screening Area. The proper design of screen chambers has an important bearing upon the success of the screening process. Screen chambers should be placed as near as possible to the surface of the ground in order to facilitate handling the screenings. They should furthermore be located near a pumping station or near the headquarters of the man in charge of the plant, in order to facilitate inspection and cleaning. They should be well ventilated, and the screens themselves should be placed at such an elevation that no serious damage would result if they should accidentally become so clogged as to obstruct the flow of sewage. Increased loss of head and consequent backing up must be considered in screen design as a factor in the load to be provided for and it is sometimes desirable, when the results of backing up would be serious, to provide for a by-pass to guard against it. Screen chambers, intentionally or unintentionally, sometimes serve also as grit chambers or detritus tanks, in which case the sludge must be regularly removed.

The design of a screen chamber is intimately related to the area of screen used, and this in turn depends upon the character and amount of sewage to be treated, the size of the screen, and especially upon the amount of attention which is to be given to keeping the screen clean. Speaking generally, for the ordinary bar or slat screen, the total open space should be somewhat greater than the cross section of the main sewer. Ogden (1908) advises making the free area 50 per cent greater than the cross

section of the sewer, which would usually mean making the total screen area about 300 per cent greater than such cross section. Raikes (1908) states that a width of 9 inches per 1000 population is the usual basis of calculation for large works; although this must depend upon the nature of the sewage, fineness of screens and maximum flow.

The following table shows the sizes of screens used in various Massachusetts municipalities, the statistics referring in all cases to screens of the fixed bar type.

TABLE XIX

SCREEN CHAMBER DATA FOR MASSACHUSETTS TOWNS

(Mass., 1904.)

Town.	Average sewage flow, gal. per day.	Open space in screen (inches).	Screening area (sq. ft.)	Amount of screenings removed.
Andover	125,000	$\frac{1}{2}$	18	$\frac{1}{2}$ cu. ft. per week.
Brockton	878,000	$\frac{3}{4}$	100	185 lbs. per day.
Clinton	785,000	$\frac{1}{2}$	117.5	
Framingham	652,000	$\frac{3}{4}$	60	Wheelbarrow load twice a week.
Gardner *	250,000	$1-\frac{1}{2}$	3.88	
Natick	566,000	$\frac{3}{4}$	22	One bucketful in two weeks.
Pittsfield	1,456,000	1″ 1st screen $\frac{3}{4}$″ 2d screen	30 ea.	2 wheelbarrow loads per day.
Southbridge	350,000	$\frac{3}{4}$	56	
Spencer	375,000	$\frac{3}{4}$	70	4 wheelbarrow loads daily.
Stockbridge	75,000	$\frac{3}{4}$″ 1st screen $\frac{1}{2}$″ 2d screen	42, 1st 8, 2d	2 wheelbarrow loads a day.
Westborough	282,000	$\frac{1}{2}$	20	

* Templeton System.

In Figures 24 and 25 are shown a general view and certain details of the screen chamber at Washington, Pa., which, fortunately, could be placed near the surface of the ground so that the screenings can be readily removed by means of a rake, placed in wheelbarrows and hauled to the area chosen for final disposal.

In planning a screening plant it is most essential to provide in some way for the prompt and effective disposal of the screenings; for it must be remembered that they are composed to a consid-

erable extent (depending on composition and length of travel of sewage) of solid fecal matter and their decomposition is of a highly offensive nature.

Efficiency of Coarse Screens. On account of the practical difficulties incidental to collecting truly representative samples of unscreened sewage and to determining, by analytical methods, the exact amount of suspended matter therein, it is difficult to

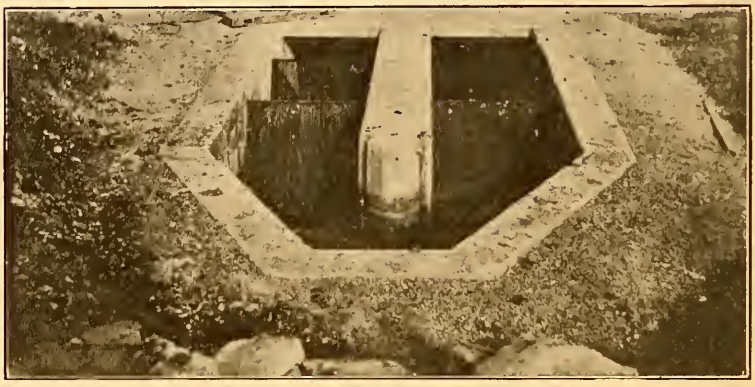

FIG. 24. View of Screen Chamber at Washington, Pa.

discuss the efficiency of screens, in terms of per cent removal, as is done in connection with tanks or filters receiving screened sewage.

Such data as are available generally refer to the amount of solid screenings produced, rather than to any comparison of screened and unscreened sewage. These data are expressed sometimes in units of weight and sometimes in units of volume, and the relation of these two units is highly variable. Kuichling (1909), in discussing the efficiency of certain mechanical screens, assumes that one cubic foot of wet screenings (83 per cent moisture) weighs 30 pounds. Bredtschneider (1905) assumes that a cubic foot of wet suspended matter weighs 23 pounds; and Monti found this volume (with 56 per cent moisture) to weigh 20 pounds.

Considerably higher estimates, based on experience in this country, are given by Johnson (1905), who reports that at the Columbus experiment station, the wet screenings weighed 65 pounds per cubic foot; and W. M. Brown, Chief Engineer of the

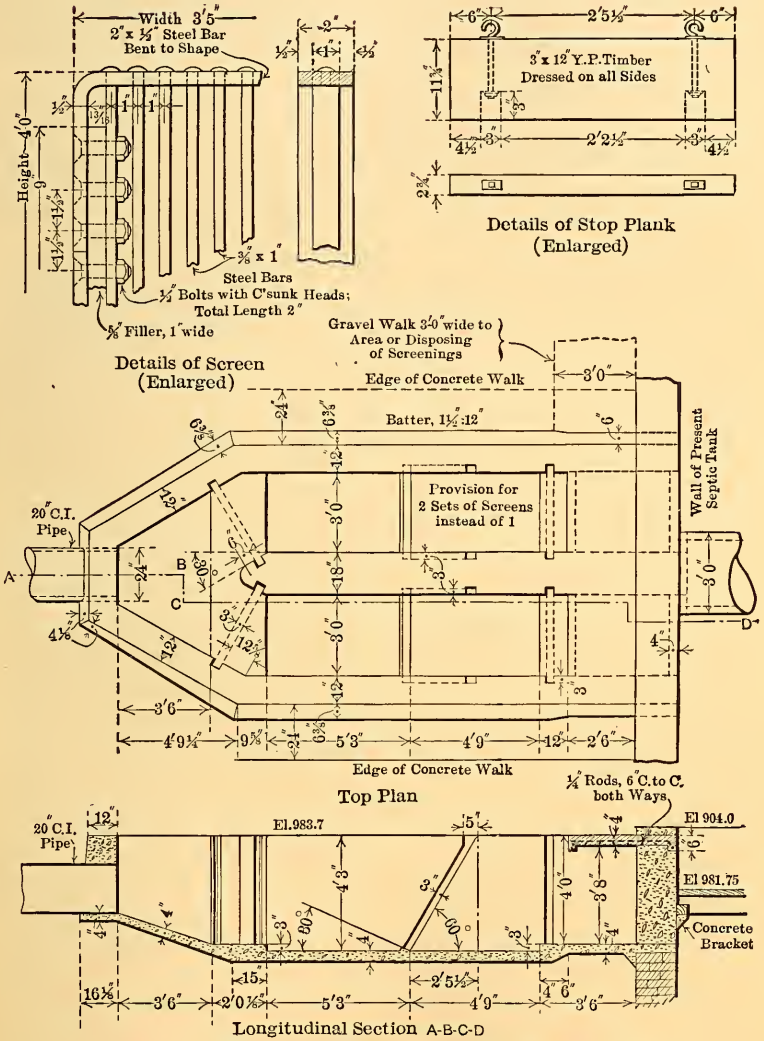
Width 3'5"
2" x ½" Steel Bar Bent to Shape
Height 4'0"
3/8" x 1" Steel Bars
½" Bolts with C'sunk Heads; Total Length 2"
⅜" Filler, 1" wide
Details of Screen (Enlarged)
3" x 12" Y.P. Timber Dressed on all Sides
Details of Stop Plank (Enlarged)
Gravel Walk 3'0" wide to Area or Disposing of Screenings
Edge of Concrete Walk
Batter, 1½":12"
Provision for 2 Sets of Screens instead of 1
Wall of Present Septic Tank
20" C.I. Pipe
Edge of Concrete Walk
Top Plan
¼" Rods, 6" C. to C. both Ways
El. 983.7
El 904.0
El 981.75
Concrete Bracket
Longitudinal Section A-B-C-D

FIG. 25. Detail of Screen Chamber at Washington, Pa.

Metropolitan sewerage works at Boston, estimates the weight of wet screenings at 60 pounds.

The table below (Table XX) shows the results attained at the Boston Metropolitan screen chambers expressed in cubic feet and gives a fair idea of what can be done with fixed bar screens of large opening.

TABLE XX

AMOUNT OF SCREENINGS REMOVED IN THE METROPOLITAN SEWERAGE DISTRICTS OF MASSACHUSETTS

Year.	North District.		Year.	South District.	
	Average daily quantity passing screens (million gals.)	Screenings removed (cu. ft. per million gallons sewage screened).		Average daily quantity passing screens (million gals.).	Screenings removed (cu. ft. per million gallons sewage screened).
1904	57.2	2.7	1905	25.0	5.9
1905	54.4	2.7	1906	33.6	6.5
1906	58.1	2.8	1907	40.6	5.0
1907	64.3	2.8	1908	37.8	4.3
1908	59.8	2.8	.....	.....	.....

The screens of the North district are of $\frac{3}{4}$-inch bars set $1\frac{3}{4}$ inches center to center, giving an open space of 1 inch in the clear. At the pumping stations of the South district there are double screens, the arrangement of which is described on page 69.

Data for a coarse bar screen with bars 4 inches apart in the clear at Milwaukee (Milwaukee, 1915) show an annual average of 0.5 cubic foot or 0.018 ton per million gallons of sewage treated. Monthly averages range between 0.015 yard in June and 0.024 yard in October. The corresponding sewage flow was 589.6 million gallons of sewage in June and 452.8 in October.

At Worcester (Worcester, 1913) a half inch bar screen produced 8.8 cubic feet per million gallons of sewage treated (varying from 6.2 in December to 13.1 in March). The total weight of the screenings averaged 440 pounds per million gallons, of which 173 pounds were dry solids, and 134 pounds organic matter.

Efficiency of Fine Screens. The figures tabulated on page 87 are given by E. H. Beard for the Reading plant, where the screen has 40 meshes to the inch.

The fresh screenings at Reading contain 90 per cent of moisture and the dried screenings only 75 per cent. Since ordinary

bar screenings contain about 75 per cent of moisture the true effect of screening at Reading is to take out about four times as much solid material as is removed by the coarse gratings of the South Metropolitan district of Boston.

TABLE XXI

AMOUNT OF SCREENINGS REMOVED BY REVOLVING SCREEN AT READING, PA.

(Beard, 1909.)

Month.	Screenings dried by centrifugal machine, screenings per million gallons.		Month.	Screenings dried by centrifugal machine, screenings per million gallons.	
	Pounds.	Cubic feet.		Pounds.	Cubic feet.
February	810	.....	August	.....	15.66
March	570	.....	September	.....	30.24
April	930	.....	October	.....	18.90
May	1020	.....	November	.....	19.17
June	1300	.....	December	.....	18.36
July	1390	.....			

Further data for other screening plants are given in the table below.

TABLE XXII

EFFICIENCY OF SCREENING WITH SCREENS OF VARIOUS GRADES

(Johnson, 1905. Kuichling, 1909.)

Place.	Average daily quantity passing screens, gallons.	Size of screen openings, clear.	Quantity removed per million gallons.	
			Cu. ft.	Pounds.
Providence, R. I.	19,500,000	1 inch	.....	41
Boston, North	57,800,000	1 inch	2.8	168
Berlin	58,749,000	$\frac{1}{2}$ inch	10.6	212
Coventry	.....	$\frac{2}{5}$ inch	.....	250–300
Göttingen	.....	$\frac{2}{5}$ inch	.....	336
Columbus Testing Sta.	350,000	$\frac{1}{2}$ and $\frac{3}{8}$ inch	4.6	300
Boston, South	37,800,000	$\frac{1}{8}$–1 inch	4.3	258
Leeds	52,000	$\frac{1}{8}$ inch	5.5	.....
Dresden	.....	$\frac{1}{12}$ inch	23.1	.....
Wiesbaden	.....	$\frac{1}{2}$, $\frac{1}{5}$–$\frac{1}{8}$ and $\frac{1}{25}$ inch	36.9	.....
Reading	2,040,000	$\frac{1}{50}$ inch	41.0	1000

In general, it appears that screens with an open space in the neighborhood of $\frac{1}{2}$ inch or thereabouts will take out from 3 to 5 cu-

bic feet of screenings per million gallons, while with clear openings of $\frac{1}{12}$ inch or less the material removed may rise to 20–40 cubic feet per million gallons. Kuichling (1910), assuming that wet screenings weigh 30 lbs. per cubic foot and contain 75 per cent of moisture, calculates the removal in parts per million at the places tabulated above as follows: Coventry, 7.5–9.0; Göttingen, 10.1; Reading, 90.0; Dresden, 177.8; Wiesbaden, 274.5.

Monti (1903) in a careful study of Berlin sewage passed through experimental screens of various openings from $\frac{7}{25}$ to $\frac{1}{50}$ inch found that the whole series took out between 10 and 14 per cent of the total dry suspended matter in the sewage. Of the total suspended matter removed by the five screens the $\frac{7}{25}$-inch screen took out 46 per cent, the three intermediate screens 24 per cent, and the $\frac{1}{50}$-inch mesh 30 per cent. In experiments at Philadelphia a conical screen of red metal cloth with 32 meshes to the inch effected an average reduction in suspended solids from 200 to 133 parts per million (Philadelphia, 1911), an efficiency of 33 per cent. Figures of 63 per cent for Chicago sewage passed through a band screen, and of 25–28 for Cleveland sewage treated by small experimental screens have been cited above. George M Wisner reports a removal of 9–25 per cent of the suspended matter in Packingtown sewage by screens with 30–40 meshes to the inch (Chicago, 1914), and we have quoted the removal of only 4 per cent effected by the Riensch-Wurl screen at Cleveland.

A very valuable study of the efficiency of screens of different mesh in treating a dilute and a very concentrated sewage was made by Langdon Pearse, the general results being summarized in Fig. 26. It will be noted that very fine screening proved of little or no advantage with the dilute sewage.

Among the most extensive studies recently made upon fine screening were those conducted at the sewage experiment station of the city of New Haven, Conn., under the direction of one of the writers (C.-E. A. W.). In these tests rectangular screens of wire cloth and perforated metal plates were placed in an inclined position in tanks designed for the purpose and the sewage was allowed to flow through until the head had built up to about 2 feet (which took from 20 to 50 minutes). A test of this sort is somewhat too favorable for the screen since it permits the accumulation of a heavier mat than would occur in practice. Yet

the results obtained, with a fresh and rather dilute day sewage, were not very striking. In 22 tests with East Street sewage a 30-mesh wire cloth screen removed from 8 to 21 per cent of the suspended solids (average, 15 per cent). Two tests with this

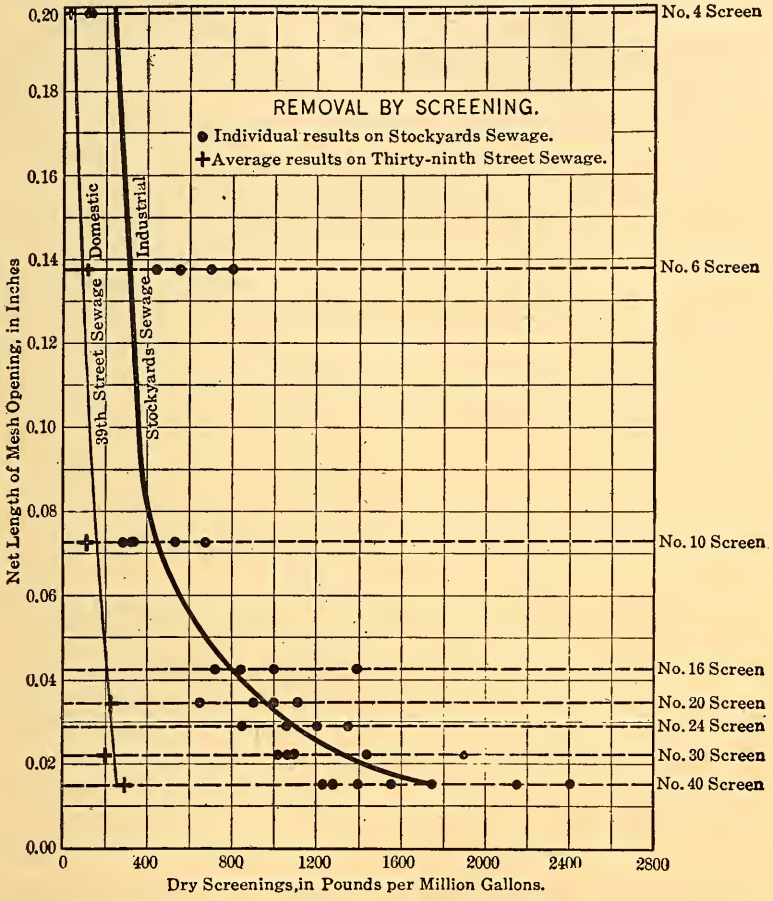

FIG. 26. Relation between Size of Screen and Weight of Screenings Produced (reproduced by courtesy of Langdon Pearse).

same screen on the sewage from another main sewer (the Boulevard sewer) gave an average removal of 12 per cent of suspended solids. A plate with slots $\frac{1}{16}$ inch wide and $1\frac{1}{4}$ inches long gave with the East Street sewage from 7 to 11 per cent removal of suspended solids (average of 6 tests, 8 per cent).

With the Boulevard sewage this plate screen took out only 3 per cent of suspended solids.

At Rochester, N. Y., Riensch-Wurl screens have operated very successfully, producing however only about 6 cubic feet of wet screenings per million gallons of sewage treated, a value not very different from that obtained with coarse bar screens.

Cost of Screening. Little authentic information is available in regard to the cost of screening sewage. With small and moderate-sized plants, the cost may be very slight, for the reason that no additional attendance is required. In other cases the cost of screening may form an appreciable item.

Below is stated the cost of labor in operating screens in the Massachusetts Metropolitan Sewerage District. The large amount of sewage pumped at these stations makes necessary the employment of men who do nothing but take care of the screens, and cost records are very carefully kept. The table brings out clearly the relative decrease of cost with an increase of work where the 8-hour day is in force.

TABLE XXIII

COST OF LABOR FOR OPERATING SCREENS AT CERTAIN STATIONS IN THE BOSTON METROPOLITAN SEWERAGE DISTRICT

(1908)

Station.	Average daily material screened, million gallons.	Size of screen opening.	Cost per million gallons.
Deer Island	59.8	1 inch	$0.14
East Boston	57.8	1 inch	0.14
Charlestown	31.3	1 inch	0.24
Ward Street	22.3	$\frac{1}{8}$–1 inch	0.53

At the Deer Island, Charlestown and East Boston stations, respectively, three screen men are employed.

The cost of fine screening will naturally prove much higher. Allen (1915) cites a cost of $1.00± for Reading, and figures ranging from $0.32 to $3.42 for various German installations. Metcalf and Eddy (1916) cite costs ranging from $0.93 to $1.69 at Birmingham. The Cleveland estimates cited above would amount to $3.20 per million gallons.

The Use of Strainers or Roughing Filters. The screening or straining of sewage through coke or similar materials, for the removal of the coarse suspended matter, has been studied experimentally to a considerable extent, and has been tried in a few instances on a practical scale. The suspended matter which is removed from the sewage rapidly fouls the material of which the strainer is made, *i.e.*, coke, coal, slag cinders, peat, etc., so that this material, especially the top portion of it, has to be frequently removed and disposed of, either by drying and burning for fuel under boilers, or by some of the methods used for the disposal of screenings. In construction, a strainer is similar to a filter. The action which takes place in the former, however, is, in general, simply a mechanical one and independent of biological or chemical agencies.

At the Lawrence experiment station of the Massachusetts State Board of Health experimental studies of coke strainers using "Station sewage" were begun in 1894. At first the material used consisted of a layer of coke breeze about 6 inches in depth, containing more or less fine material. It was found necessary to remove about 10 cubic yards of this material for every million gallons of sewage strained. The depth of the strainer was increased to 12 inches and it was operated for some three years, with the result that 8 cubic yards of material had to be removed for each million gallons of sewage strained.

On account of the fine dust in the material it was often found that clogging occurred in the underdrains. Later, therefore, there was used a strainer 15 inches in depth, containing coke from which dust and fine material had been removed. This was operated for over two years, during which time it was necessary to remove only $\frac{4}{10}$ of a cubic yard per million gallons of sewage strained. The strainers above described were operated at a rate of about 1,000,000 gallons per acre per day. The removal of organic matter was 34 to 50 per cent, on the basis of oxygen consumed.

The experiments at Columbus, Ohio (Johnson, 1905), included studies of coke strainers and the results obtained were very similar to those recorded at Lawrence. The general effect of the process is indicated in the table below. It was found impossible to burn the clogged coke without first subjecting it to a thorough drying process, and drying was accompanied by the

production of extremely offensive odors. Furthermore, the efficiency of the strainers fluctuated considerably, as a result perhaps of breaks occurring in the straining surface.

TABLE XXIV

EFFICIENCY OF COKE STRAINING AT COLUMBUS, OHIO

(Johnson, 1905.)

Tons of dry solids per acre.

	A	B
In applied sewage	265	263
Removed by strainer	169	213
Subsequently removed from surface of beds	45	49
Subsequently found in the body of the strainer	117	151
Presumably dissolved through septic action	3	10
Passed out in the effluent	96	50

Perhaps the most instructive attempt to use strainers on a practical scale has been made at Gardner, Mass., where coke strainers were installed for clarifying sewage, before treatment on intermittent sand filters. The quantity of sewage treated daily amounted to about 575,000 gallons and represented a tributary population of 9000. The coke strainers comprised four units of $\frac{1}{8}$ acre each, the total area being $\frac{1}{2}$ acre. They were constructed of an 8-inch layer of coke breeze (effective size, 0.41 mm.), supported by 6 inches of broken stone graded from $\frac{1}{2}$ to 1 inch. Underdrains were laid 6 feet apart in the stone. It was planned to distribute the sewage over the coke from perforated wooden troughs and an automatic apparatus was installed to dose the strainers on the contact bed principle. This apparatus was never very successfully operated, and was only used for a few years.

The sewage was afterwards run onto two of the four strainers for a week or so at a time. It stood about a foot deep over the coke, and while a portion was strained the larger portion overflowed through a pipe discharging into the underdrains — provided for the purpose when the strainers first failed to work properly. The two strainers used were rested about once a fortnight, and the sewage was diverted to the other two. Drying took several days, and it was often necessary to use the beds again before they were cleaned. The strainers failed to work in

winter, and became easily clogged in the spring and summer months.

Conclusions in Regard to the Value of Screening. Coarse screening is obviously an essential step in very many, indeed in most processes of sewage treatment. The extent to which fine screens are likely to be used in the United States remains on the other hand somewhat problematical. In any given case the principal factors to be considered will be the amount of solid matter which it is necessary to remove, the amount which can be removed by screening, the reliability of the apparatus from a mechanical standpoint, the opportunities for disposing of the screenings to be produced, and the cost of construction and operation of screens as compared with the cost of tank treatment and the liability to local nuisance.

It may be noted that the results to be expected from screening will vary widely with the character of the sewage treated, a strong fresh sewage being much more favorable than a weak and stale one. The higher dilution of American sewage makes it unreasonable to expect such favorable results here as have been obtained in Germany.

Allen (1915) estimates that a removal of 30 per cent of suspended solids may generally be expected from screens of 0.1 inch mesh or under, and holds that by the best fine screening we may look for a removal of 30 to 50 per cent as compared with 50 to 65 per cent for sedimentation. Fuller (1912) too is optimistic stating that " screens have a far greater field of usefulness than is generally recognized at present in America." On the other hand the experiments at Chicago led Langdon Pearse to believe that a 30 per cent efficiency was far too high to be generally counted on, and the Cleveland and New Haven studies lead to a similar conclusion. Metcalf and Eddy (1916) conclude: "It seems probable that the day of elaborate fine screening has passed, in view of the results attained with improved sedimentation tanks. If the matter removed by fine screening can be successfully taken out in settling basins, the cost will usually be less."

As a matter of fact it seems to us more than doubtful whether fine screens operating with an average American sewage, can ever be expected to compare with efficient clarification from the standpoint of removal of suspended solids. Even Mr. Fuller, who has

been the leading advocate of fine screening in recent years, does not, as we understand, claim that they will remove as a rule more than 10 per cent of the suspended solids in American sewage. On the other hand there are many cases in which a removal of the coarser floating particles, offensive to sight, is all that is required prior to disposal by dilution. Under such conditions screens may offer great advantages over tank treatment on account of compactness of construction and relative freedom from nuisance (provided that arrangements are made for the prompt and regular removal of the screenings which are produced). Particularly where the presence of industrial wastes would be likely to interfere with the successful operation of the Imhoff tank (as at Indianapolis and New Haven) the use of fine screens as an alternative to tank treatment may sometimes be desirable.

CHAPTER IV

PRELIMINARY TREATMENT OF SEWAGE BY SEDIMENTATION

General Objects of Sedimentation. One of the most serious problems in sewage disposal is presented by the suspended solid material, partly mineral and partly organic in nature. The nitrogenous organic matter tends to form foul deposits when sewage is disposed of by dilution, and both organic and inorganic suspended solids through their clogging action seriously increase the cost of all systems of disposal by filtration. Where disinfection is to be provided for it is important to eliminate large particles into which disinfectants will not penetrate. To some extent in almost all plants, and to a great extent in highly developed and specialized plants, the physical factor of sedimentation is invoked to remove the heavier portions of this suspended matter before the true processes of chemical and biological purification are brought into play.

Sedimentation is conditioned by the velocity of the liquid in which it takes place, by the size and specific gravity of the particles involved and by the time during which the force of gravity is allowed to act. In discussing the theoretical process of sedimentation with respect to the clarification of turbid river waters Hazen (1904) has constructed a diagram showing the relation between a given period of sedimentation (in terms of time required for one particle to settle from top to bottom) and the corresponding percentage of particles remaining in suspension. This mathematical discussion was of necessity based on very broad assumptions, one of which was that all particles of sediment in the water have the same hydraulic subsiding value; *i.e.*, settle at the same rate.

The sedimentation of the suspended matter in sewage is susceptible of much less accurate mathematical analysis for the reason that the particles of such suspended matter are constantly changing in shape and hydraulic subsiding value, as a result of the marked bacteriological and chemical changes which take

place. The ebullition of gas is another factor which may exert a material influence upon the process. A considerable fraction of the suspended matter in sewage (normally about one-half) is in a state of very fine (colloidal) division and cannot be removed by any process of unaided sedimentation.

The general method relied upon in sewage sedimentation is of course a reduction of the velocity of flow by passage through a chamber of larger dimensions than the sewer itself; and the most important fundamental variables are the total capacity of this chamber, governing the storage period, and its width, governing the velocity of flow. Fuller (1909) points out that when sewage is disposed of by dilution the velocity in the sedimentation basins should be at least as low as that in the currents into which the effluent is to be discharged. In preparation for filtration the process should be carried far enough to remove as much of the suspended matter as can be disposed of more cheaply in this manner than by discharging it on the filters.

Types of Sedimentation. Sedimentation tanks, using the term in the broadest sense, may be designed to deal only with the grosser and heavier particles or they may be planned so as to remove a large proportion of the finer suspended organic matter as well. According to this difference in aim we may distinguish between Grit Chambers and true Sedimentation Tanks.

In the construction of sedimentation tanks there is again a radical distinction in plan between two types, the shallow tanks, with mainly lateral flow, and the deep tanks (of which the Dortmund tank is an example), with mainly an upward flow.

Finally there are important modifications in the operation of sedimentation tanks which practically constitute independent processes. If chemical precipitants are used to accelerate the action we have Sedimentation Aided by Chemicals. If the sludge, instead of being frequently removed, is allowed to remain in the bottom of the sedimentation tank so that a considerable portion of it is liquefied by bacterial action, the tank is known as a Septic Tank, — or an Imhoff Tank if a separate lower sludge digestion chamber is provided.

In connection with sedimentation tanks there should also be mentioned the Slate Beds of Dibdin. These slate beds, though usually classed as filters, are in fact made up of a series of small

sedimentation tanks (each having a capacity of only a few cubic inches) in which the solid matter from the sewage is deposited and allowed to undergo aerobic bacterial changes.

The present chapter deals only with the various forms of Grit Chamber Treatment and Plain Sedimentation. Chemical Precipitation will be discussed in Chapter V, Septic Tanks in Chapter VI and Imhoff Tanks in Chapter VII. The Dibdin Slate Bed, although practically a method of preliminary treatment for the removal of suspended solids, will be taken up later, in Chapter X, for the reason that it was historically an outgrowth of the contact process and because the theory of the contact bed is necessary for a proper comprehension of its mode of action.

Grit Chambers or Detritus Tanks. Grit chambers are used for the purpose of removing from the sewage, by sedimentation, the coarser and heavier suspended material of mineral character. Such material consists largely of surface washings from streets, and hence grit chambers are most necessary where the sewerage system is on the combined plan. In fact their use with a system of strictly domestic sewers may cause an objectionable accumulation of foul organic matter which could more economically and satisfactorily be taken care of elsewhere. Grit chambers are often installed in connection with screening apparatus. They should be of relatively small capacity, and the velocity of flow should be sufficient to prevent the deposition of organic matter; at the same time, however, it is important, especially if subsequent sedimentation tanks are to be operated on the septic plan, to have the grit chamber large enough to remove the sand and road detritus which would otherwise rapidly fill the sedimentation tanks and interfere with the decomposition of the sludge by bacterial action. A proper velocity of flow is therefore highly important. Where, as in the case of precipitation tanks, the sludge is to be regularly removed and pressed, the thorough preliminary removal of detritus is not so important, as the presence of a little grit in the sludge facilitates pressing. On the other hand, too much sand and gravel, mixed with the sludge, may make it impossible for the main body of sludge to flow by gravity; and furthermore, this material may injure sludge pumps, should such be used.

In order to meet these requirements, the dimensions of a grit

chamber are generally such as to allow for a storage period of one to two minutes, with the minimum velocity reduced as a rule to about one foot per second. The chamber should be designed so that the detritus may settle as evenly as possible along the bottom, and not form a heavy deposit at the inlet end and leave but little at the outlet. All grit chambers should be in duplicate, in order that one may be put out of service for cleaning without interference.

A good type of grit chamber, shown in Fig. 27, was installed in 1904 at the Worcester Massachusetts sewage plant. It con-

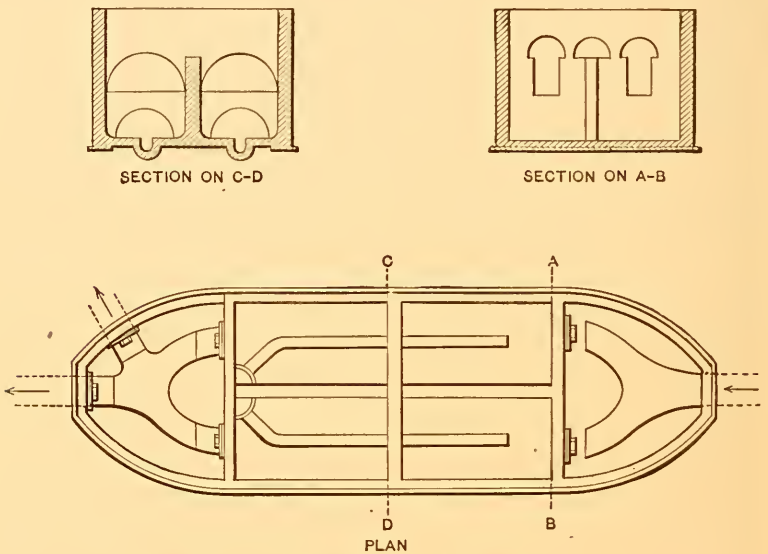

Fig. 27. Grit Chamber at Worcester, Mass.

sists of two parallel channels, each of which is 40 × 10 feet, and 9 feet deep. One-half of the structure can therefore be placed out of service for cleaning.

The accumulated detritus at Worcester is removed by shoveling, and this is the common practice in the United States. In many of the German and English works the material is dredged out by mechanical means (see Fig. 17) and at several places in Germany perforated metal vessels are fitted to the interior of the tanks and hoisted by cranes when full.

Based on data from the Cleveland Sewage Testing Station,

the engineers at Cleveland designed a grit chamber having adjustable vanes, which, when in operating condition, gave the tank a resemblance to a two-story tank without the trapped slot. This permitted the deposited grit to descend to a low compartment so that its accumulation did not influence the cross section available for the flowing sewage. The adjustability of the vanes also, of course, permitted one to vary the cross section and hence control the velocity of the flow within certain limits. As an auxiliary feature, one of the tanks was designed so that the grit would drop into hopper bottoms from which it flowed into ejectors which could be operated without draining the grit chamber.

Quantity of Detritus. The amount of detritus varies very widely with the amount of surface water admitted to the sewer system and is markedly influenced by storms. At the Dorchester pumping station of the Boston Main Drainage works, in 1907, there was removed from 32 billion gallons of sewage 578 tons of screenings and 5306 cubic yards of fine detritus. The latter, the detritus alone, works out at 0.17 cubic yard per million gallons. Assuming for the screenings a weight of 60 pounds per cubic foot, the total amount of screenings and detritus together would be 0.19 cubic yard per million gallons. Metcalf and Eddy (1916) give the average amount of grit intercepted at the Worcester plant from 1905 to 1913 as 0.125 cubic yard per million gallons, the annual averages for this period ranging from 0.20 in 1905 to 0.09 in 1913.

At the Technology Experiment Station in Boston a detritus tank was arranged to give a velocity of 12 mm. per second (2.4 feet per minute) and careful records of the amount of sediment were kept. The total detritus removed by the grit chamber amounted in sixteen months to 4800 pounds, or 2.4 cubic yards, of wet material from a total volume of sewage equal to a little over 3 million gallons which is equivalent to 1600 pounds, or 0.65 cubic yard per million gallons. All the material thus removed was carefully sampled and its moisture determined. A portion of each dried sample was preserved and mixed with proportionate parts of later samples, and the mixture was finally analyzed. The amount of moisture, the proportions of clean stone and of dry detritus, and the analysis of the latter, are shown in Table XXV, first in total amounts and then in parts per million of the total volume of sewage (3 million gallons).

TABLE XXV

AMOUNT AND COMPOSITION OF DETRITUS REMOVED FROM GRIT CHAMBER AT BOSTON, MARCH 26, 1904, TO JUNE 1, 1905

(Winslow and Phelps, 1906.)

	Wet detritus.	Water.	Clean stone, etc.	Fine, dry detritus.			
				Total.	Loss on ignition.	Organic nitrogen.	Oxy. cons.
Total pounds	4800	1300	570	2900	319	6.6	5.1
Pounds per million gallons of sewage	1600	430	190	970	106	2.2	1.7
Parts per million parts of sewage	190	52	23	117	13	0.26	0.2

The figures show only the total and average amounts of detritus collected for the whole period. No accurate data are at hand to indicate the effect of the seasons on the amount of detritus deposited. The Boston city records, already referred to, show that at Moon Island there is a maximum of deposit in March, and a second maximum during the summer months. Local conditions of rainfall largely determine the amount of detritus sent into the sewer, but monthly variations from the mean yearly deposit are not as a rule greater than 25 per cent of that value. At the Technology Experiment Station, in the spring of 1904, a large amount of snow was thawed by the warm rains, and during ten days 1600 pounds of detritus were taken from the detritus chamber and from a storage tank into which some excess of detritus had been carried over. This gave an average of 16,000 pounds of detritus per million gallons of sewage, and for shorter periods the rate of deposit might be greater still.

At Manchester, England, where the combined sewers are not provided with catch basins, the detritus has sometimes amounted to 300 tons in a single day after a heavy rain.

The Sedimentation of Finer Suspended Solids. The line of demarcation between the so-called detritus and the remaining suspended solids, or sewage sludge proper, is rather sharply marked by the rapidity with which the particles settle. One class of material will settle out in a very few minutes when the velocity is still considerable, and the other will settle only when the liquid is practically at rest and in the course of hours rather

than minutes. According to figures given by Robinson (1896), a velocity of 0.5 foot per second will not move fine clay and 0.7 foot will just move coarse sand. In practice a somewhat higher velocity (1.0 foot per second) has generally been found advisable for grit chambers in order to avoid offensive accumulations of organic material. The grit chambers at Rochester, N. Y., illustrate the advantage of this procedure, being designed to give a velocity of 1 foot per second and yielding an effluent containing only about 10 per cent of organic matter. On the other hand, sedimentation of the true suspended solids necessitates a slackening of velocities to 0.1 foot per second or less. In the London settling basins velocities of 0.07 foot are maintained; at Manchester, England, 0.05; at Saltley and Sutton, England, 0.03; and at Frankfurt, Germany, 0.01 to 0.02.

Important investigations in regard to the sedimentation accomplished with various rates of flow have been carried out by Bock and Schwarz at Hanover, Steurnagel at Cologne, Watson at Birmingham and Johnson at Columbus, Ohio.

Bock and Schwarz experimented in 1899 with tanks 162 and 243 feet long at velocities ranging from 0.014 to 0.027 foot per second. At a velocity between 0.014 and 0.027 foot per second they succeeded in depositing 55.7 per cent of the suspended solids in the shorter and 61.5 per cent in the longer tank. On increasing the velocity to 0.067 foot the efficiency of the longer tank was only decreased to 57 per cent. Complete quiescence for 24 hours produced a reduction of 88.8 per cent.

Steurnagel (1904) found that velocities less than 0.07 foot per second permitted results as good as any that were economically attainable. Complete quiescence for 12 hours only removed 84 per cent of the suspended solids: at a velocity of 0.014 foot per second 72.3 per cent was removed, and at a velocity of 0.070 foot, 69.1 per cent. On the other hand a further increase to 0.140 foot diminished the reduction to 58.9 per cent.

It is clear that while a good proportion of the solid matter in sewage will settle out quite rapidly, a certain fraction is too finely divided to be removed in this way under any practical conditions. The same thing was indicated by Steurnagel's studies of quiescent sedimentation. In some experiments on this point in a 40-cm. cylinder the removal of suspended solids by sedimentation was about 25 per cent in five minutes, 50

per cent in thirty minutes and 75 per cent in twenty-four hours. Steurnagel studied the same phenomenon in a deeper layer (2 meters) and found the removal of suspended organic matter to be 42 per cent in five minutes, 61 per cent in twenty-five minutes, 75 per cent in six hours and 80 per cent in twenty-four hours.

With continuous-flow tanks the storage period must of course be somewhat longer than with quiescent sedimentation; and longer tanks are necessary with high than with low velocities. The suspended matter is acted upon by two forces, the force of gravity and the force due to the velocity of the moving sewage. The resultant of these two forces, shown graphically, should intercept the inside of the tank at a point below the outlet. It is probable that practice has erred in the past on the side of using too slow velocities and too long storage periods. Present opinion in this country tends to favor a detention period of not over two hours with a velocity of 0.2 to 0.5 inch per second.

As indicated by the table below, a more rapid rate of flow has the advantage that the sludge deposited at the higher velocities contains the heavier solid matters, with less moisture, or, in other words, is more condensed than the sludge deposited at low velocities, and hence may be more easily handled.

TABLE XXVI

COMPOSITION OF SLUDGE DEPOSITED AT DIFFERENT VELOCITIES

(Dunbar, 1908.)

Velocity in feet per sec.	Sludge (gallons).	Analysis of sludge.	
		Moisture (per cent).	Dry residue (per cent).
0.014	4.040	95.57	4.43
0.070	2.474	92.87	7.13
0.140	1.838	91.34	8.66

General Design of Sedimentation Tanks. Sedimentation tanks in the past have been operated either on the fill-and-draw or continuous-flow plan. With intermittent subsidence, where tanks are filled with sewage and then allowed to remain full in a quiescent state, there is danger of stirring up the sludge

at each emptying and filling of the tank; it is necessary to use a floating or movable outlet, which is frequently unsatisfactory; and there is sometimes absorbed by this method an available head which could be used to better advantage. Continuous subsidence is therefore now usually employed.

The principal features to be considered in the design of a sedimentation tank are total capacity, relative dimensions, especially depth, and number of units. Minor details of importance are the slope of bottom, the design of floor and walls, inlets and outlets and baffles.

The capacity of sedimentation tanks has in the past often been such as to provide a detention period equal to from 4 to 12 or even 20 hours flow. Recent experience as suggested above tends materially to shorten this period. The Philadelphia experiments (Phila., 1911) showed that a reduction of storage time from 6 to $4\frac{1}{2}$ hours did not decrease the efficiency of sedimentation and that in a baffled tank $3\frac{1}{2}$ hours was sufficient. Experiments at Chicago (Chicago, 1914) gave the results indicated below.

TABLE XXVII

RELATION BETWEEN SEDIMENTATION PERIOD AND EFFICIENCY

(Chicago, 1914.)

Strength of sewage p.p.m. total suspended solids.	Per cent removal of total suspended solids.			
	1 hour.	2 hours.	4 hours.	12 hours.
1775	85	89	90	92
915	71	75	77	78
560	53	56	56	57
236	55	62	70	79
156	45	54	63	70
72	23	31	43	59

Imhoff recommends a detention of only 2 hours and Fuller (1912) suggests 2.5 to 4 hours. Fuller places the desirable limits of linear velocity between 0.17 and 0.50 inch per second, and Imhoff holds that the velocity is a matter of comparative indifference so long as it is below 1 to 2 inches per second.

Except when used for special purposes, shallow sedimentation tanks are usually built rectangular in both plan and section, as this form affords the greatest economy of construction

consistent with efficiency. By first deciding upon the total capacity and velocity of flow, the shape of the tank is largely determined; that is, there must be a definite cross-sectional area in order to correspond with the given daily flow and a given velocity. It remains, however, to decide upon the proper depth, which in turn determines the width. The depth is sometimes controlled by local topography and facilities for disposing of the sludge; but for plain sedimentation tanks depths of 8 to 12 feet are, in America, considered good practice, the deeper tanks being used when they are to be operated on the septic plan. In England a depth of 4 to 8 feet has been recommended by engineers of authority, in the belief that a more even distribution of flow will be obtained in a tank of 8 feet or less. Since relatively rapid velocities are desirable, the length of the tanks should be at least four times the width. Metcalf and Eddy (1916) cite ratios for English and German plants ranging from 1:1.7 to 1:16; at Mt. Vernon and N. Attleboro, Mass., the ratios are respectively 1:2 and 1:2.4, at Baltimore 1:4 and at Reading, Pa., 1:4.9. Long tanks involve unduly expensive construction, but high velocities and long lengths of travel can be obtained by constructing several parallel units to be used in tandem.

In deciding upon the number of tank units to be provided, fluctuations in flow should first be considered. If the dry-weather flow is very greatly increased at times by surface water, a tank which affords the proper sedimentation period and velocity of flow in dry weather will be entirely too small in times of storms, with the result that the " critical velocity " will be exceeded and suspended matter will be carried out with the effluent. By installing a number of units, some may be held in reserve for the storm flows; or, if all of the units are used in tandem during dry weather, the storm flows could be handled by operating all in parallel.

The advantage of dividing tanks into units is particularly apparent where purification works are planned to allow for a future increase in population. In such cases the sewage flow will be very small at first and will increase gradually during the first few years that the system is used. Another advantage of separate units lies in the facility for cleaning and the avoidance of large quantities of sludge to be handled at one time. All

works, no matter how small, must have at least two units, and plants of considerable size should have many more.

Details of Tank Construction. Both experiment and theory indicate that a tank for plain sedimentation should be deeper at the inlet end, in order to provide space for the deposited material without unduly decreasing the cross section of the tank. The

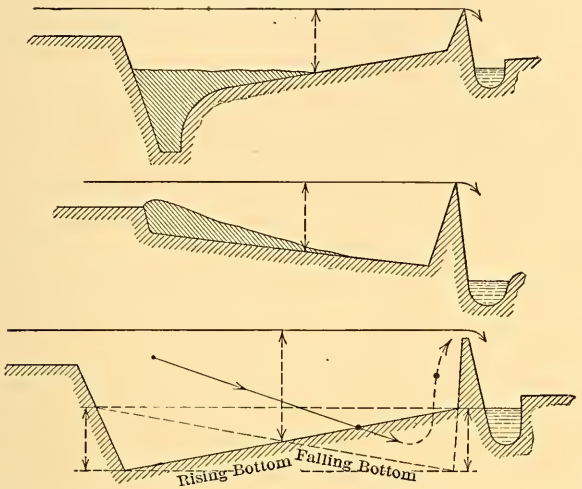

FIG. 28. Diagram of Efficient and Faulty Tank Construction (after Steurnagel, 1904.)

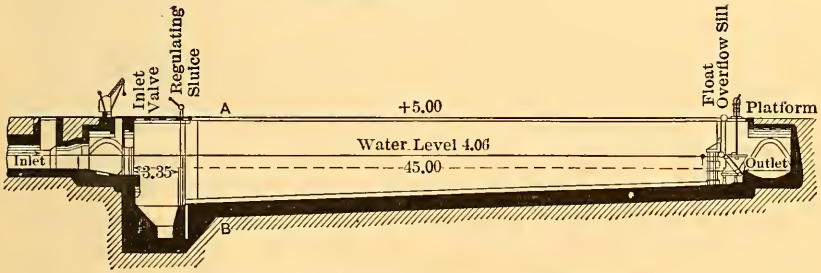

FIG. 29. Sedimentation Tank at Cologne (after Steurnagel, 1904).

stirring up of the solids deposited in the tank by temporary increase in velocity of flow is also avoided by such design (see Figs. 28 and 29).

The floors of sedimentation tanks are almost universally built of concrete, as this material may be readily adapted to the desired shape of the floor. The cost of concrete is less than

that of any other satisfactory material. Usually a thickness of 6 inches will afford a sufficiently water-tight floor, although in some cases, where the ground is soft, it may be desirable to increase the thickness or to use steel reinforcement. The durability of concrete where exposed to sewage has been questioned in certain instances, when there have been reported instances of the disintegration of concrete tanks, presumably from the action of the sewage gases. (Eng. News, 1910.)

In order to facilitate the removal of sludge, the floor should have a slope of at least 1 in 60 toward an outlet provided for draining the tank, or toward outlets for draining the subdivisions. Even with the slope above mentioned, it will be found impossible to drain the tank completely without pushing out some of the sludge by hand or flushing it with a hose stream. Such a procedure, however, is entirely practicable; and as slopes steeper than 1 in 50 tend to make the construction of the tank more difficult they are not recommended.

The design of the walls of sedimentation tanks is governed largely by the character of the foundation, the height above the original ground level and the length of the wall between supports. It may sometimes be desirable to coat the inside of the walls with pitch or tar. It has been suggested that this will not only add to the water-tightness, but will tend to protect the concrete from any action of the sewage.

In general the walls should be proportioned according to the same principles that govern the design of dams and retaining walls. A tank wall, when the tank is empty, will constitute a retaining wall if placed below the original ground level or if banked with earth on the outside. The materials of construction are usually plain concrete or reinforced concrete. With all open tanks the tops of the walls should be wide enough for a man to walk upon safely, and to attain this end a coping may be used if necessary.

The simplest form of inlet and outlet for all kinds of sedimentation tanks is a weir with perfectly level crest, extending across the tank. With this form, and especially if the tank is to be operated on the septic plan, there should be scum-boards immediately in front of both inlet and outlet to prevent surface currents. The channel conveying the sewage to the inlet weir

should not be unduly large or objectionable deposits may result before the sewage reaches the tank.

With large plants this weir construction is open to objection from a constructional standpoint and, in such cases, an even distribution of the current is obtained by discharging the sewage into the tank, and drawing it off, through several openings, generally below the surface, and by placing a baffle or deflector within a few feet of the end walls.

Submerged walls and suspended baffles, or scum-boards, for preventing the forward movement of suspended matter form an important part of sedimentation tank construction. The design of baffles requires considerable skill and judgment. Experiments at Philadelphia (Phila., 1911) showed that the results of sedimentation may be very materially improved by judicious baffling; but on the other hand, if not carefully arranged, baffles and scum boards may do more harm than good by increasing velocity to an excessive degree and stirring up the deposited sediments.

Submerged walls serve to subdivide the tank in such a manner as to facilitate the removal of sludge. They also guard against temperature effects by preventing direct currents between inlet and outlet. Schmidt has demonstrated, by means of coloring matter, that during the cooler portions of the year the warm sewage tends to remain on the top of the cooler contents of the tank.

Removal of Sludge. A point of great importance in the construction of sedimentation tanks is the provision of some means for the removal of the sludge. This must be accomplished at rather frequent intervals, particularly in summer, if putrefactive decomposition is to be avoided. In hot weather it may be necessary to remove sludge every six or seven days, while in cold weather it may be stored for a month or more. Some arrangement must be made for drawing down the supernatant liquor to the level of the sludge deposit so as to keep at a minimum the volume of sludge to be handled. This may be accomplished by means of arms on the outlet pipe so adjusted that the sewage will at all times be drawn from a level a few inches below the surface (see Fig. 30). Another device for accomplishing the same purpose consists of a vertical standpipe or outlet chamber provided at every 1 or 2 feet with openings, controlled from

above, through which the tank contents may be drawn off at given levels.

When the tank effluent is to be discharged intermittently, on sand or other filters, it may be desirable (in order to avoid building a dosing tank) to install some automatic controlling

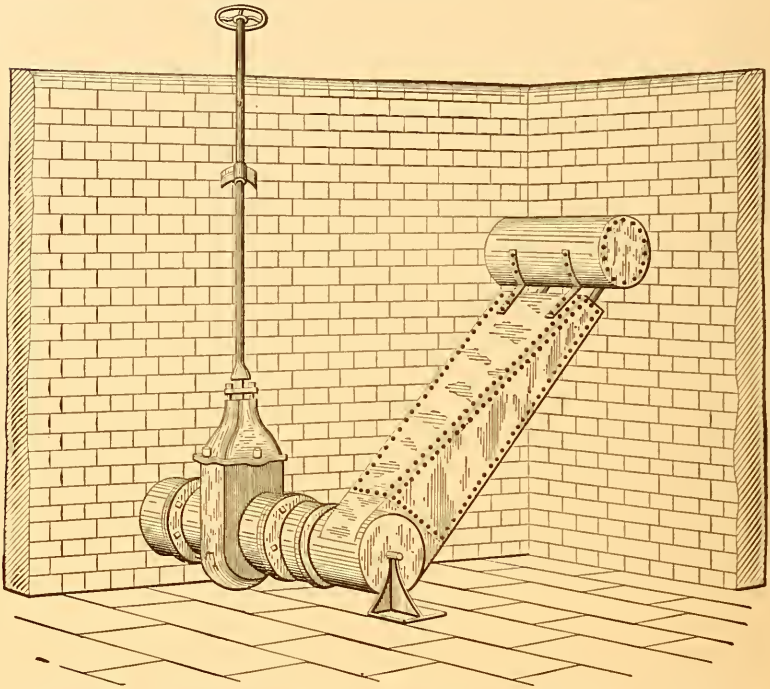

Fig. 30. Floating Arm for Emptying Sedimentation Tank (copied by permission from Santo-Crimp, 1894).

device which will hold back the flow until the sewage level has risen to a certain height and then allow it to discharge at a rate greater than the rate of inflow. With such a device the sewage, after rising in the tank to the desired level, feeds into a float chamber through a siphon. This causes the float to rise and open a butterfly valve controlling the main outlet of the tank. When the sewage level drops the float lowers and closes the outlet.

In any case the sludge is finally allowed to run off from a special pipe at the lowest point and the heavier sludge must

generally be worked down to the outlet by hand. At some of the English plants interesting mechanical appliances are installed for moving sludge to the tank outlet. A device of this sort in use at Bolton is described in Chapter V.

Sedimentation in Deep Tanks. An entirely different type of sedimentation basin is the deep tank with a conical bottom, in which the sewage flows upward and the sludge is removed from below while the tank is in continuous operation. The first important tank of this kind, intended for use without the application of chemicals, was built at Dortmund, Germany. For

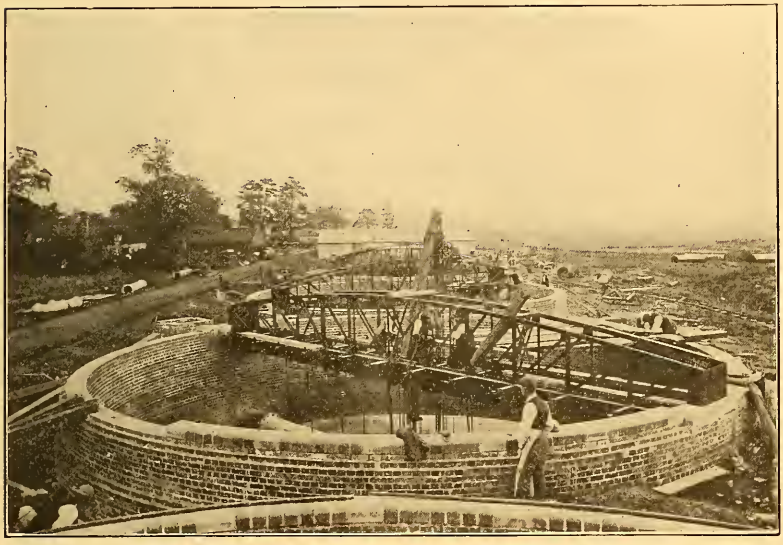

FIG. 31. General View of Deep Sedimentation Tank at Birmingham, England (courtesy of J. D. Watson).

this reason the name " Dortmund " has been applied to tanks of this design.

The principal advantage claimed for this type of tank lies in the fact that sludge may be removed without stopping its operation. On the other hand, the sludge drawn off from such tanks is very liquid. Deep tanks are generally more expensive to construct than shallow ones; and in their practical operation it has been found difficult to prevent the sludge from adhering to the sides of the conical bottom. Frequent cleaning is necessary if

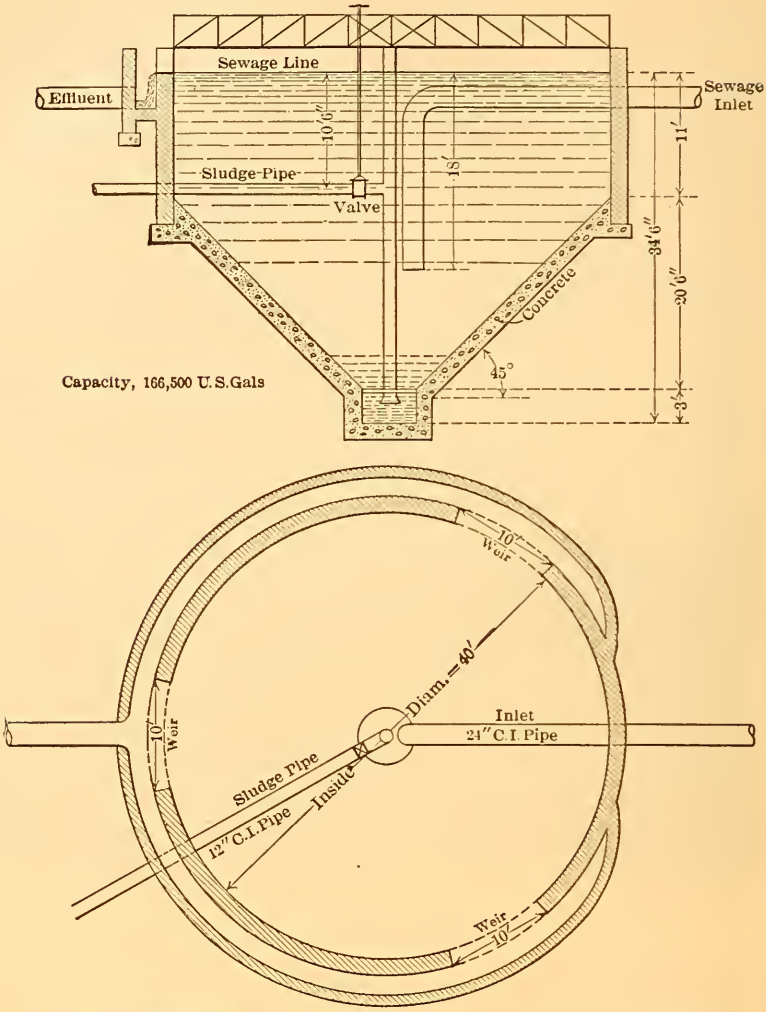

FIG. 32. Plan and Section of Deep Sedimentation Tank at Birmingham, England (courtesy of J. D. Watson).

the tank is to be successful, and in England various kinds of revolving scrapers have been designed for this purpose.

At Birmingham two sets of deep tanks are used, for treating the septic effluent before filtration and the trickling effluent after filtration. The first tanks (receiving septic effluent) are cylindrical above and conical below, as shown in Figs. 31 and

32. The sludge collects at the bottom of the cone and there enters a pipe, inside which it is forced up and out, when the sludge valve is opened, by the weight of the liquid in the tank.

The sewage is admitted by a vertical pipe at the center with its open end near the bottom, and the settled sewage overflows by weirs near the top of the tank.

Tanks of the vertical flow type have been used in the United States for the treatment of the sewage at the World's Fair at Chicago in 1893 (Hazen, 1894); for the treatment of sewage from an insane hospital at Kings Park, N. Y. (Fuller, 1912); and for the treatment of a strong industrial sewage at Gloversville, N.Y. (Vrooman, 1910). Experiments at Chicago (Chicago, 1914) showed about the same results (50–70 per cent removal of suspended solids) from vertical and horizontal flow tanks. Great care was necessary however with tanks of the former type to prevent accumulation of solids and subsequent septic decomposition. Experiments at Cleveland (Hoffmann, Pratt and Hommon, 1914) gave rather unsatisfactory results with vertical flow tanks.

Efficiency of Sedimentation. The efficiency of sedimentation will vary widely with the composition of the sewage and with various conditions of construction and operation.

It is obvious that the depth of the tank will be a very important factor if it is inadequate and of little moment when beyond a certain limit. So too the arrangement of inlets, outlets and baffles may have a very harmful effect if faulty, but cannot improve efficiency beyond a certain point when they are properly designed. As to the effect of the composition of sewage, the data in Table XXVIII from George A. Johnson's Columbus studies is very instructive as showing how a high percentage removal is favored by strength and low organic content of the sewage treated.

The rate of settling of fine particles at different temperatures varies as $\frac{t + 10}{60}$ when t is the Fahrenheit temperature, so that the rate of settling at 74 degrees will be twice that at the freezing point.

A considerable fraction of the suspended matter in sewage is in a state of such fine division that it cannot be removed by any process of plain sedimentation, unless aided by chemical pre-

cipitation or by some other process of removal by adsorption. Such non-settleable matter is generally referred to as "colloidal matter" using the term in an empirical sense. The most careful studies which have been made on this point, at Columbus, Ohio, showed that 34 per cent of the suspended solids present were in this state as shown by their failure to settle out in eight

TABLE XXVIII

EFFICIENCY OF SEDIMENTATION WITH SEWAGE OF VARYING STRENGTH AND VARYING ORGANIC CONTENT

(Johnson, 1905.)

	Percentage of total suspended solids of organic nature.			Percentage removal of suspended solids.		
Character of sewage.	Strong.	Medium.	Weak.	Strong.	Medium.	Weak.
July 11–12.....	54	50	35	61	52	35
July 12–13.....	51	46	47	70	60	19
July 13–14.....	39	54	50	75	58	17

hours (Johnson, 1905). At Philadelphia (1911) 36 per cent of the suspended solids were of colloidal nature. Metcalf and Eddy (1916) cite data obtained by Bugbee showing 45 per cent of colloidal solids in Providence sewage and by Hommon showing 19 and 49 per cent respectively at the two Atlanta plants. Vigorous agitation tends to increase the proportion of suspended matter in the colloidal state. O'Shaughnessy (1914) cites data from Leeds which showed 30 per cent of solids as colloidal matter before and 42.5 per cent after pumping.

In a few cases very high efficiencies are reported for sedimentation tanks in actual use. At Birmingham the deep vertical flow tanks remove 85 per cent of the suspended solids from a trickling filter effluent, and Dunbar (1908) states that 80 to 90 per cent of the suspended matter in sewage can be removed. On the other hand, most English and American data do not indicate that any such results can be obtained with crude sewage. According to Fuller (1909): "From 50 to 65 per cent of the suspended matters in well-screened sewage can be removed by sedimentation in basins holding 6 to 12 hours, average flow. . . ."

The efficiency of a number of continuous-flow shallow sedimentation tanks in England and America is indicated in the

table below, the original data being given in the Final Report of the Royal Sewage Commission and in G. W. Fuller's Cornell address:

TABLE XXIX

EFFICIENCY OF SEDIMENTATION TANKS

Place.	Sedimentation period, hours.	Rate of flow, inches per minute.	Suspended solids, parts per million.		Per cent reduction.
			Influent.	Effluent.	
Halton, Eng.	15.5	0.66	177	107	40
Clifton, Eng.	5.3	1.70	490	240	51
Oswestry, Eng.	4.1	3.40	320	158	51
Dorking, Eng.	11.9	0.31	208	101	51
Plainfield, N. J.	10	.....	118	54	54
Columbus, O.	13	.....	304	101	67
Reading, Pa.	15	.....	165	42	75

In general it may be concluded that about 50 per cent of the solid matter in sewage may be practically removed by plain sedimentation; and this degree of purification can usually be attained by a comparatively brief treatment, usually not exceeding two hours.

The sludge accumulating in sedimentation basins amounts to 4 to 6 cubic yards per million gallons with American sewage; and the disposal of this sludge, which forms a separate and very serious problem, will be discussed in Chapter XIII.

CHAPTER V

PRELIMINARY TREATMENT OF SEWAGE BY CHEMICAL PRECIPITATION

History of Chemical Precipitation. The failure of sedimentation to clarify sewage thoroughly, and the hope of financial gain, led to the introduction in England, during the early sixties, of what is known as the chemical precipitation of sewage. It was claimed that by this method the putrefactive substances in sewage would be removed, and that the sale of the precipitated nitrogenous compounds as a fertilizer would pay for the cost of treatment and realize a substantial profit. The process was at first greeted on all sides as a method which would solve the sewage problem, and between 1880 and 1890 nearly all the sewage plants installed in England were chemical precipitation plants.

The hope of financial gain, however, was not realized, for though the precipitate contained nitrogenous compounds and a certain amount of phosphate, it also contained so much water — averaging about 94 per cent — that it was found impossible to prepare a marketable product at a cost at all comparable with the price of other chemical fertilizers. Furthermore it was gradually recognized that though the process yielded a clear liquid, the liquid contained a large amount of soluble organic matter and was still putrescible.

Notwithstanding these facts, however, the process continued in general use into the nineties, when the bacterial methods of purification, by which non-putrescible effluents could be obtained, began to receive serious attention from sanitary engineers.

The chemical treatment of sewage consists in adding to the sewage certain chemical compounds which cause a voluminous flocculent precipitate to be formed, and allowing the sewage thus treated to remain quiescent in a large tank for a number of hours, or to flow continuously through a series of tanks at so slow a velocity that the insoluble compounds formed, together

with the suspended matters of the sewage, settle to the bottom of the tank, leaving a clear supernatant liquid.

The effect of the precipitant is to carry down a considerable fraction of the so-called colloidal matter which cannot be removed by the action of sedimentation alone. Thus the experiments at Chicago (1914) showed a removal of suspended solids amounting to 50 to 70 per cent by plain sedimentation, while a removal of 80 per cent was effected by sedimentation aided by chemicals.

Precipitants Used in Treating Sewage. The chemical compound first used was lime, added to the sewage in the form of milk of lime. The action of the lime depends upon its uniting with the free and combined carbonic acid in the sewage to form insoluble calcium carbonate, which in settling out of the liquid carries down with it the suspended matter. The amount of lime used at different places has varied, depending on the character of the sewage, an acid sewage requiring much more lime than a neutral or slightly alkaline sewage to produce a given effect. The best results were obtained by the addition of just sufficient lime to decompose the carbonates present in the sewage, *i.e.*, from five to ten grains of lime per gallon. If too little lime was used, sedimentation took place very slowly, and a clear effluent could not be obtained; if too much lime was added, varying amounts of the suspended organic matter in the sewage were rendered soluble, causing the effluent to be more putrescible and more obnoxious than the effluent from plain sedimentation.

Besides lime, various other chemical compounds have been tried, resulting in innumerable patents being taken out for the chemical treatment of sewage. Of these chemical compounds, lime and ferrous sulphate, or lime and aluminium sulphate, have been more extensively used than any others, on account of simplicity of application and superior effectiveness under ordinary conditions.

Experiments as to the effect of lime, iron salts and aluminium sulphate on sewage have been made by Hazen in America and Dibdin in England. Hazen (1890), working with the sewage of Lawrence, Mass., which contained but little trade waste, was alkaline to litmus paper, and gave on analysis in parts per million 18.3 parts ammonia nitrogen, 2.7 parts albuminoid nitrogen

in suspension and 3.9 parts in solution, found that lime, ferrous sulphate and aluminium sulphate, when used in the correct amounts, removed practically all the suspended matter, and, judging from the albuminoid nitrogen, from 20 to 40 per cent of the soluble organic material (including some colloidal matter). He found that a definite amount of lime gave a result as good as, or better than, either more or less lime; that the more copperas, ferric sulphate or aluminium sulphate used the better the result; that no lime was necessary with aluminium sulphate or ferric sulphate if the sewage was alkaline; that with lime alone the best results were obtained when the amount of lime added was just sufficient to decompose the carbonates present; that with either ferric or aluminium sulphate 3.5 grains per gallon, or 500 pounds to the million gallons, gave nearly as good results as larger amounts; that with ferrous sulphate and lime there was no great advantage in using more than 7 grains per gallon, or 1000 pounds of the iron salt per million gallons; that much better results were obtained by first adding lime and then copperas than by the reverse process; that ferric sulphate was preferable to ferrous sulphate, not only because it could be used without lime, but because the ferric hydrate was more insoluble than the ferrous hydrate and brought about quicker sedimentation.

Dibdin (1889), working on London sewage, obtained the best results with lime and copperas, 10 grains of each to an English gallon, all the suspended matter and 30 per cent of the soluble organic matter being removed. Lime used alone, 15 grains to the gallon, removed 25 per cent, and lime and alum, 5 grains of each per gallon, removed 18 per cent of the soluble organic matter.

The above experiments show that lime with either ferrous sulphate or alumimium sulphate, when used in proper amounts, will remove practically all the suspended matter and a certain amount of the soluble organic matter when tested in the laboratory and though the removal when working on a large scale is less than indicated by the above experiments, the general results are in accord with those obtained by Hazen and Dibdin.

It should, however, be stated that though iron salts and lime effect a more complete removal of organic matter than aluminium sulphate and lime, the effluent obtained from the use of

iron salts is in certain respects less satisfactory than that obtained from aluminium salts. It always contains a certain amount of iron in solution, as iron forms soluble salts with organic acids which are not decomposed by the action of lime. The presence of iron salts in an effluent causes more or less trouble, especially when the effluent is further treated by bacterial methods. The soluble organic iron compounds undergo decomposition with the final formation of insoluble ferric oxide, which not only clogs the sand of a bacterial filter bed, but may close up the open joints of the underdrains.

Among the other chemical processes used may be mentioned the Spencer Alumino-Ferric process which consists in adding alumino-ferric, a mixture of ferrous and ferric salts with aluminium sulphate, alone or with lime, to the sewage; the International process which consists in adding a compound called "ferrozone" to the sewage and filtering the effluent through "polarite"; the A. B. C. process, at one time very prominent as one of the chemical precipitation methods, which was supposed to consist in adding to the sewage, alum, blood, and clay, though as generally used, the substances were alum, clay, and carbon, 50 grains of the mixture per gallon of sewage; and the Webster process in which the active precipitant was ferric hydroxide, obtained by passing a weak electric current through iron plates suspended in the sewage channels, the plates being connected alternately with the positive and negative terminals of a dynamo.

General Practice in Chemical Precipitation. Taking into account all the known facts, it would seem that the chemicals best adapted for the treatment of sewage are ferric sulphate and lime. Aluminium sulphate is theoretically preferable to ferrous sulphate, as it does not tend to form soluble salts with the organic matter; yet, in general practice, ferrous sulphate is ordinarily used, for as a rule it can be obtained more easily and cheaply than either of the other two salts.

The amount of ferrous sulphate added to sewage free from trade waste seldom exceeds 5 grains per gallon, although experiments have shown that the best results are obtained when the amount reaches 7 grains per gallon. In sewage containing trade waste, the amount of iron present is often so great that any further increase is unnecessary, and all that is required is the addition of lime. The amount of lime used depends on the

character of the sewage. The test usually applied at sewage works to determine whether the right amount of lime has been introduced is to add a few drops of an alcoholic solution of phenolphthalein to the sewage. If the right amount of lime has been added a very faint pink color is produced; if not enough lime has been added no color is formed; if too large an amount, a very decided red or pink color appears. The amount of lime and iron or aluminium salts added varies widely at different places on account of the differences in the character of the sewage.

At London 3.3 grains of lime and 0.82 grain copperas have been used for one United States gallon; at Salford, 7.5 grains lime and 2.5 grains copperas; at York, 4.7 grains lime and 3.6 grains alumino-ferric; at Bolton, 7.83 grains ferrozone; at Leeds, 2.8 grains lime; at Sheffield, 4.2 to 6.7 grains lime; at Providence, R. I., 5.08 grains lime; at Worcester, Mass., 6.57 grains lime. One grain per gallon equals 17.1 parts per million.

The lime should be added to the sewage in the form of milk of lime. A soft fine-grained lime containing over 80 per cent of calcium oxide, which slakes easily, should be selected. It should be thoroughly slaked, and if possible a day's supply should be weighed out and slaked twenty-four hours before its intended use. Sufficient water or sewage should be added to the slaked lime to form milk of lime, as otherwise part of the slaked lime settles out of the liquid and is rendered inactive. When either copperas or aluminium sulphate is used, it should be dissolved in weighed amounts in a known volume of water contained in vats, so that the amount added to the sewage can be easily regulated by adjusting the flow of the liquid. The copperas or the aluminium sulphate should be added to the sewage below the point where the milk of lime enters, and to insure a thorough mixture of the chemicals with the sewage, the sewage should be run through a channel containing baffle boards before it enters the tank.

The chemical reactions that take place are as follows: the milk of lime, calcium hydroxide, first reacts with the free carbonic acid and the acid carbonates contained in the sewage, insoluble calcium carbonate being formed, and if mineral acids are present it neutralizes them and renders the sewage alkaline. When the copperas or aluminium sulphate is added to this alkaline sewage the excess of calcium hydroxide decomposes

the salt with the formation of ferrous or aluminium hydroxide and calcium sulphate. The calcium carbonate and the ferrous or aluminium hydroxide formed are heavy insoluble substances, which in settling out of the sewage carry down the suspended matter, and by occlusion, or by the formation of insoluble colloids, a small amount of the organic soluble substances as well.

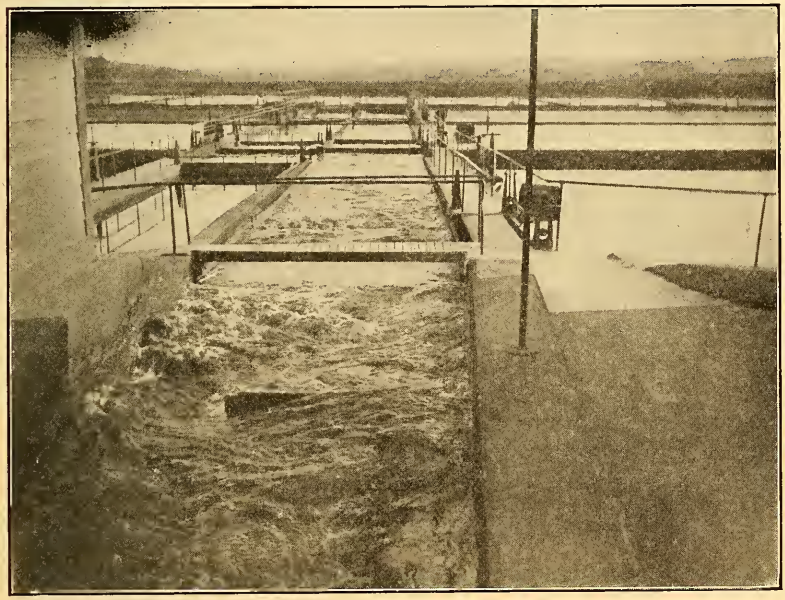

Fig. 33. Mixing Channel with Baffle Boards, Worcester Sewage Works.

Construction of Tanks. Both the shallow rectangular tanks and the deep Dortmund tanks, described in Chapter IV, are used in the chemical treatment of sewage, and as chemical treatment is practically only sedimentation aided by the addition of chemicals, what has been said regarding the construction and use of settling tanks in the chapter on Sedimentation applies to tanks used for chemically treated sewage. The Dortmund tanks, however, have not as yet been employed for this purpose, either in England or America, but in Germany they are considered especially adapted for handling the sludge from chemical precipitation.

In the early days of chemical precipitation it was customary to allow the sewage after the addition of chemicals to remain quiescent in a tank for six hours, then to draw off the clear liquid and again fill the tank with sewage. There is, however, as has been stated in Chapter IV, a serious objection to this method, in that it is almost impossible not to stir up the sludge when the tank is emptied or filled, unless the sludge is removed each time the tank is emptied. Consequently, this procedure has been generally superseded by the method of continuous flow, and though very little experimental work has been done to determine the rate of flow best adapted to sedimentation when chemical substances have been added to the sewage, general practice indicates that with shallow tanks a period of six to eight hours gives the most satisfactory result.

Formation and Removal of the Sludge. The amount of sludge formed by chemical precipitation is at least fifty per cent more than the amount formed by plain sedimentation, although it depends more or less on the character of the sewage and the chemicals used.

At London about 953 cubic feet are produced per million gallons of sewage; at Salford, 1233; at Leeds, 820; at Sheffield, 383.5; at Providence, R. I., 525; at Worcester, Mass., 659 cubic feet.

The sludge is a slimy mass with a specific gravity of about 1.04 to 1.06 and must be removed from the tanks before active putrefaction sets in, as otherwise the gas formed brings to the surface of the liquid a greater or less amount of the deposited substances, which are then carried away in the effluent. This fact, in warm weather, often necessitates the removal of the sludge once in every seven to ten days. For the purpose of removing the sludge from shallow tanks the supernatant liquid is run off by the use of a floating arm, or by means of a series of superimposed valves as described in Chapter IV. Though the sludge contains 90 to 95 per cent water, only a portion will flow of itself to the lower end of the tank or through the sludge channel, which is often made in the cement concrete of the bottom of the tank, and the flow must be assisted by the use of hand or horse power propelled squeegees. (Fig. 34.)

The Ashton Mechanical Squeegee. At Bolton an ingenious mechanical device, known as the Ashton mechanical squeegee, is

FIG. 34. Hand-propelled Squeegee used at Salford, England.

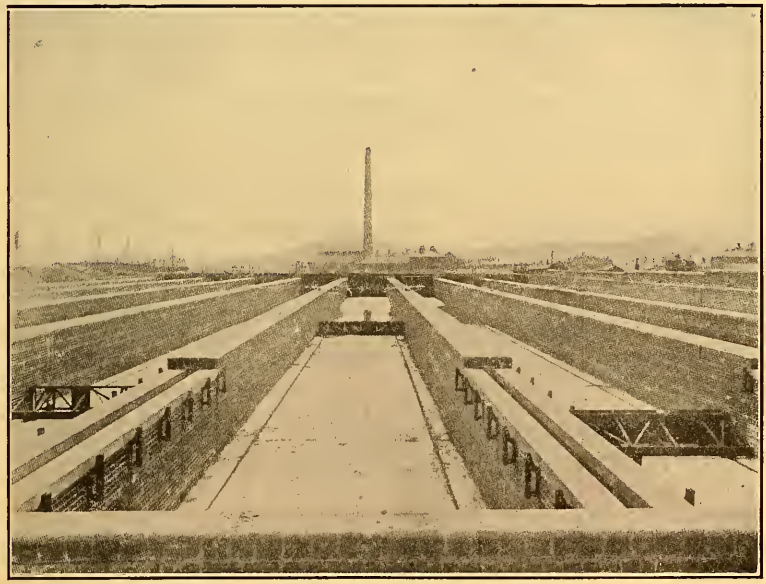

FIG. 35. Ashton Mechanical Squeegee (by permission of Mr. Ashton).

used. (Fig. 35.) It is constructed of angle iron, having grooves along the front into which one-inch boards can be placed. The bottom boards and ends have facings of rubber. The machine is run on two light iron tracks set in the concrete at the bottom of the tank. To operate the mechanical squeegee successfully the supernatant liquid is drawn off down to the sludge by means of a floating arm or other valve device. Sufficient clarified sewage from the neighboring tank is then run in behind the machine to move both the sludge and machine to the bottom of the tank, the fall being 1 in 80. To move the squeegee back to the upper end of the tank, sewage is run into the clean tank, and as the volume increases sufficient power is obtained to push the machine backwards to its original position at the top end of the tank, where it is again ready for removing the sludge.

With deep tanks it is not necessary to remove the liquid, the sludge being drawn off from the bottom of the tank, or raised through an iron pipe by the pressure of the overlying liquid, or by pumping. There is more or less difficulty, however, in preventing the adhesion of some of the solid matter to the sides of the tank, leading to putrefaction which may give more or less of a septic character to the liquid. The sludge obtained by chemical treatment of sewage has the same characteristics and properties as the sludge from plain sedimentation or from the septic tank process, and the question of the ultimate disposal of all of these sludges will be considered in Chapter XIII.

TABLE XXX

EFFICIENCY OF CHEMICAL PRECIPITATION

Parts per million.

	Suspended solids.		Albuminoid nitrogen.	
	Sewage.	Effluent.	Sewage.	Effluent.
Leeds	775	96	7.88 Total	5.09 Total
Bolton	496	176	9.7 "	3.4 "
Bradford	840	36	32.8 "	17.80 "
Sheffield	417.5	103.4	9.02 "	5.68 "
Salford	466	30	.....	.....
York	212	89	8.2 Total	5.4 Total
Providence	397	47	8.86 " 4.28 Dissolved	4.80 " 4.15 Dissolved
Worcester	231	68	7.33 Total 3.34 Dissolved	4.66 Total 3.65 Dissolved

Results Obtained by the Chemical Treatment of Sewage. The chemical treatment of sewage removes practically only the suspended solid matter. Under favorable circumstances and with careful work a certain percentage of the soluble organic matter may be carried down, but there is no workable precipitation process which will give a non-putrefactive effluent. The amount of purification accomplished by chemical treatment is indicated in the table on p. 122.

Metcalf and Eddy (1916) tabulate data for ten English cities showing a percentage removal of suspended solids ranging from 64 per cent at York to 95 per cent at Kingston, with data for six American plants and five tests at the World's Fair and two at Columbus which range from 26 per cent removal at Glenville, O., to 83 per cent at Providence. Worcester, the only other large plant under careful operation besides Providence, showed a removal of 82 per cent.

Chemical Precipitation Plants in Great Britain. Though chemical precipitation is generally considered, only as a method of preliminary treatment, its use as a final process may in certain cases be permissible, where the effluent is discharged into so large a volume of water that the removal of suspended matter constitutes a sufficient purification.

In Great Britain, London and Glasgow are the two largest cities which use chemical precipitation as a final process; and it is also used in a number of places as a preliminary process. At Leeds the septic tank method of clarification was abandoned and chemical treatment instituted in its place.

The sewage of the city of London was originally discharged from various sewers directly into the Thames, producing an unbearable nuisance, mentioned in Chapter II. To cope with these conditions the Metropolitan Board of Works, with Sir Joseph W. Bazalgette as engineer, began in 1858 the construction of intercepting sewers on both sides of the river. The Crossness outfall sewer was completed in 1862, and those at Barking in 1863 and 1864. Covered reservoirs were built at both Crossness and Barking, one of 30,000,000, the other of 42,000,000 U. S. gallons capacity. These reservoirs were designed to store the sewage for discharge on the outgoing tide. The capacity of the reservoirs soon proved insufficient, and on account of numerous complaints as to the condition of the river, it was decided after

a long inquiry to adopt chemical precipitation as the method of treatment, using lime and copperas as the precipitants. The Barking precipitation works were put into operation in 1889 and the Crossness works in 1892.

The cost, as given by Baker (1904), of treating the sewage and conveying the sludge to sea has been about $8.67 per million U. S. gallons. As the result of the chemical treatment of London sewage 80 per cent of the suspended matter is removed. The Glasgow plant is fully described by Moore & Silcock (1909).

Chemical Precipitation in the United States. The two largest plants in the United States for the chemical treatment of sewage are at Worcester, Mass., and Providence, R. I.

At Worcester the plant is now used to treat that portion of the sewage which cannot be handled on intermittent filtration beds, the number of the latter not being sufficient to treat all the sewage. At Providence all of the sewage is treated, the effluent being discharged into Narragansett Bay. At both cities shallow tanks are used, and the sludge is pressed. At Worcester it is carried by cars to a deep isolated valley and dumped, little or no nuisance being created. At Providence it is carried by a double-bottom scow and dumped at the United States Government dumping station in 75 feet of water.

Worcester was the first large city in the United States which attempted to treat sewage before emptying it into a watercourse, and consequently the history of the plant, together with the present method of treatment, may be considered in some detail.

Treatment of Sewage at Worcester, Massachusetts. By an act of the state legislature in the year 1867, the city of Worcester, having a population of about 30,000, was granted the use of the various brooks within its limits for sewerage purposes. Accordingly, a sewer system was built upon the combined plan, emptying into the Blackstone River, a comparatively small stream with a drainage area as it leaves the city of about 60 square miles.

As early as 1880 the towns along the river below Worcester began to complain of offensive odors. The outcome was an act of the legislature requiring the city to treat the sewage before discharging it into the stream.

A small chemical precipitation plant was put into operation

in 1890 designed to treat 3,000,000 gallons of sewage daily. The works consisted chiefly of six settling basins, each having a capacity of 350,000 gallons, lime house, power plant and laboratory. In 1893 the number of settling basins was increased to sixteen, giving a total capacity of 5,500,000 gallons. The total cost of the precipitation works was approximately $200,000. This plant was capable of dealing with the entire dry weather flow of sewage and brook water, amounting to about 15,000,000 gallons daily, which was nearly the full capacity of the outfall sewer.

A sludge-pressing plant was built in 1898, and in the same year the city constructed fourteen acres of intermittent sand filtration beds. The number of beds has been increased from time to time, and at the close of 1909 there were 65.2 acres of actual filtering surface.

The sewerage works in 1916 comprised 109 miles of sanitary sewers, 69 miles of combined sewers and 61 miles of surface water drains. The average daily flow in 1916 amounted to about 20 million gallons. The sewage contains large amounts of trade waste, composed chiefly of pickling liquors from wire mills and foundries, and the refuse from tanneries. Its general character is shown by analyses cited in Chapter I.

In 1916, 78.3 per cent of the sewage flow was treated by chemical precipitation and the remaining 21.7 per cent, after a short period of sedimentation, was filtered through sand beds. All of the sewage was first passed through a grit chamber. The portion to be chemically treated was treated with milk of lime, averaging 838 pounds per million gallons. The ground plan of the precipitation works is shown in Fig. 36. The milk of lime is added to the sewage at (R) about 100 feet above the mixing channel. The sewage passes through the mixing channel, in which are baffle boards, so as to insure thorough mixing of the lime with the sewage. The flow is then divided, a part passing to the right through basins, Nos. 1, 2 and 3, and the remainder passing through basins, Nos. 4, 5 and 6. These basins serve as roughing tanks, retaining the bulk of the sludge. The partially settled sewage then flows through the finishing tanks, Nos. 7–16, in parallel, and finally into the channel (S) leading to the river. The flow through the tanks, averaging about 6 hours, is controlled by weirs across the outlets of the basins, and floating substances

are prevented from passing off in the effluent by the use of scum boards.

The sludge, containing 93 per cent water, averages 5000 gallons per million gallons of sewage. To remove this sludge from

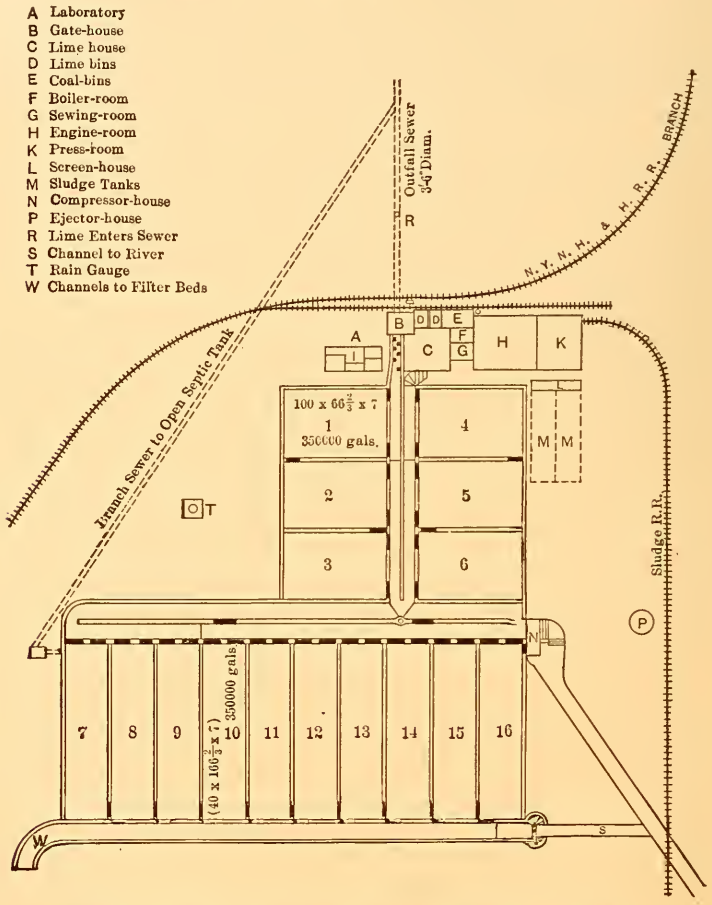

Fig. 36. Plan of Worcester Chemical Precipitation Plant.

the basins, the water is drawn off by means of floating arms, and the sludge allowed to flow by gravity, each basin containing a central sludge channel, to the sump well (P). From the sump well it is pumped to the storage tank (MM) by means of a Shone ejector, and 30 to 50 lbs. of lime in the form of milk of lime

are added. The sludge is allowed to settle in the storage tanks and the clear liquid, 15 to 25 per cent of the total volume of the sludge, is drawn off on to sand filter beds.

The storage tanks are provided with vertical bar screens (L)

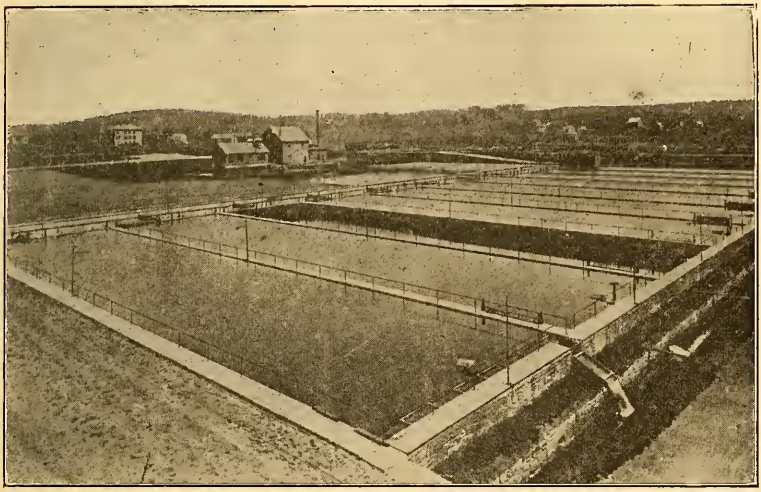

FIG. 37. Bird's-eye view of Worcester Chemical Precipitation Plant.

with half-inch openings, and the screened sludge is pumped by power plunger pumps into the filter presses under 80 pounds' pressure per square inch.

The liquor is conducted to sand filter beds, and the sludge cake, containing 70 per cent water, is hauled by motor cars to an isolated valley three-quarters of a mile from the press house.

The removal of the suspended organic matter by the above process, measured by albuminoid nitrogen, has in the past averaged about 85 to 90 per cent, which is roughly 50 per cent of the total organic matter in the sewage; but in 1916 the purification amounted only to 71 per cent of the suspended, or 38 per cent of the total nitrogen, on account of a great excess of iron sulphate in the sewage and a shortage of lime. The cost of chemical treatment in 1916 was $4.15 per million gallons and the cost of sludge pressing $7.05 per million gallons.

The sewage to be purified by intermittent filtration, 5 million gallons per day, after passing through the grit chamber is

caused to flow through a sedimentation tank having a capacity of about one-half hour's flow. Here the grosser suspended matters are removed and disposed of by pumping onto adjacent fields.

The ordinary dose of sewage applied to the filters is from

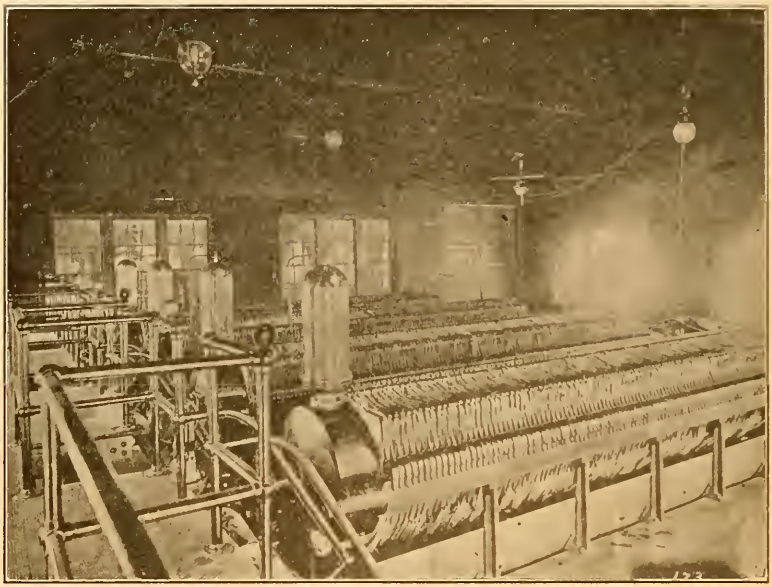

Fig. 38. Sludge Presses at Worcester Chemical Precipitation Plant.

300,000 to 400,000 gallons per bed, and the average rate per acre amounts to about 75,000 gallons daily. The accumulation on the surface of the beds is raked up late in the fall into small piles about 4 feet apart, which are left during the winter to assist in holding up the ice from the surface. In the spring the beds are given a thorough cleaning, and the surface accumulation, unavoidably mixed with some sand, is carted away and used for filling in the swamps and lowlands. The clogging matter so removed amounts to about 250 cubic yards per acre or 10 cubic yards per million gallons of sewage filtered. The cost of cleaning averages 35 cents per cubic yard.

The ferrous sulphate present in great quantities in Worcester sewage is largely oxidized during the process of filtration, and

much of it settles out as hydrated ferric oxide in the gravel around the open joints of the underdrains, eventually clogging the openings. The drains in some of the beds have been relaid twice during the last ten years. This item increases the cost of operation about 20 per cent.

The total cost of operation of the filters amounted in 1916 to $8.08 per million gallons of sewage filtered.

The entire cost of the disposal plant up to 1916 was $781,185.20. The cost of maintenance for 1916 was $57,976.36 or 34 cents per capita.

Extensive studies of alternative methods of treatment have been carried out at Worcester, septic and Imhoff tanks, contact and trickling filters all having been investigated. In 1912 it was reported that "the cost of operation of Imhoff tanks and sprinkling filters per million gallons of sewage treated would be very much less than the cost of chemical precipitation or sand filtration as carried on at Worcester." At present activated sludge treatment is being considered as perhaps still more favorable (Butcher, 1917).

Acid Precipitation of Sewage with a View to Grease Recovery. Where sewage contains large amounts of certain trade wastes, treatment with acids for the recovery of grease has been advocated as a particularly promising form of chemical treatment. At Bradford, England, for example, the sewage contains considerable quantities of wool grease, and the treatment consists in passing the sewage through detritus tanks, screening, and adding sulphuric acid, 30 to 40 grains H_2SO_4, 1.82 sp. gr., per United States gallon. The sewage is passed through settling tanks used in series. The sludge, containing 80 per cent water and 8.57 per cent grease, after removal from the tanks is treated with a further quantity of sulphuric acid, heated to 100° C. and run through filter presses. The grease obtained is said to pay for the tank treatment of the sewage.

A similar procedure has recently been advocated in this country for the treatment of ordinary domestic sewage. The process in question has been patented by George W. Miles of Boston and experiments in that city have been reported by Weston (1916). The treatment consists in "the addition of an acid, to precipitate the bulk of the solids from sewage in the form of a sludge which can be dried and degreased, thereby producing

a readily salable and greaseless fertilizer, as well as recovering the valuable grease. Either sulphuric or sulphurous acid may be used and the process contemplates the manufacture of the acid at the disposal works. If sulphuric acid were chosen, ordinary weak chamber acid of 1.53 specific gravity would be used, but the cheapest source of acid is undoubtedly pyrite (native FeS_2) which, when roasted in a furnace of proper construction, produces sulphur dioxide (SO_2). This is a gas which may be fed directly into the sewage, in which it would dissolve, forming sulphurous acid" (Weston, 1916). The sulphur dioxide is a strong disinfectant and accomplishes a material disinfection of the sewage as well as considerable clarification. The process was studied in a series of tests by E. S. Dorr of the Sewer Division of the City of Boston between 1911 and 1914 and for a week in July, 1915, and three days in November, 1915, by Prof. R. S. Weston of the Sanitary Research Laboratory of the Massachusetts Institute of Technology.

An excess of acid was added to the sewage at all times, this excess amounting to about 35 parts of SO_2 per million. The average amount of sulphur dioxide added per million gallons of sewage was 2300 pounds in the Dorr experiments and 1963 in those of Weston. The average detention period for subsidence is given by Weston as 7.7 hours. Thirteen thousand four hundred pounds of wet sludge (85.8 per cent moisture) per million gallons were obtained by Weston, giving a dry sludge figure of 1909 pounds per million gallons, closely checking Dorr's figure of 1814 pounds. The grease in the sludge amounted to 22.66 per cent of the dry sludge or 430.1 pounds per million gallons of sewage, again closely checking Dorr's figures of 22.33 per cent and 436 pounds per million gallons respectively. The fat determined by Weston in the sewage amounted only to 36.8 parts per million, while the 430 pounds in the sludge were equivalent to 51.7 parts per million in the sewage, the increase being attributed to the decomposition of soaps under the action of the acid.

Weston (1916) reports that this process reduces " the numbers of bacteria from millions to hundreds per cubic centimeter" and that " the sludge can be held at the works for four days during the heated season without giving off offensive odors, while the effluent is inoffensive and stable enough to be discharged into Boston Harbor without the slightest probability of creating a

nuisance." He cites estimates by Mr. Dorr of $18 per million gallons of sewage for the cost of treatment, while the sludge he believes to be worth $24 per million gallons, leaving a profit of $6 per million gallons. He adds in comment " while it is believed that this estimated margin of profit is too high, the writer is unable, by any reasonable comparison with analogous cost data from other sources, including his experience, to wipe it out." He concludes " with the facts at hand, the process would be very satisfactory for Boston from a sanitary standpoint and is more promising economically than any other known."

Experiments at Baltimore, Md. (Eng. News, 1916*a*), have shown that sludge precipitated from Baltimore sewage by the action of sulphuric acid contained 27.5 per cent of grease on the dry basis. Eighty-six per cent of this grease was saponifiable. The sludge contained 93.9 per cent moisture, was very offensive in odor and hard to dry, the results in these respects differing materially from those obtained at Boston.

A study of this process of sewage treatment was made in connection with the Report on Industrial Wastes from the Stock Yards and Packingtown in Chicago made to the Board of Trustees of the Sanitary District of Chicago, in October, 1914. The ether extract in this sewage is very high, averaging for the day sewage 135 parts per million. By treatment with 3200 pounds of 100 per cent acid per million gallons of sewage treated, 69 per cent of this ether extract was removed (against 47 per cent removed by plain sedimentation). The mixed sludge and scum contained 88–96 per cent moisture and 22–26 per cent of ether soluble material (the latter, on the dry basis).

The most extensive investigation of this process yet conducted has been made at the experiment station at New Haven, Conn., during the years 1917 and 1918 under the direction of one of the writers (C.-E. A. W.). Four different runs were made with the East Street sewage and one run with the Boulevard sewage, each run ranging from 24 to 75 days. The tank used at the East Street outlet was of wood, 16 feet long, 3 feet 6 inches wide, and 4 feet deep, with a capacity of 1680 gallons, and the average detention period was 4 hours. The acid was applied in the form of SO_2 gas. The alkalinity of the East Street sewage is very low (averaging only 50 parts for the four runs), so that it was necessary to add only 700 pounds of acid per million gallons

of sewage treated. The results, so far as the effluent was concerned, were admirable, as indicated by the data tabulated below.

TABLE XXXI

RESULTS OBTAINED AT NEW HAVEN SEWAGE EXPERIMENT STATION WITH MILES PROCESS OF TREATING SEWAGE

(East Street sewer.)

First run, June 2–25, 1917, inclusive, 25 days.
Second run, July 5–28, 1917, inclusive, 24 days.
Third test, Nov. 6–Dec. 19, 1917, inclusive, 44 days.
Fourth test, Jan. 18–April 4, 1918, inclusive, 75 days.

	Chlorine.	Alk'y.	Ammonia N.	Nitrite, nitrate N.	Susp'd solids.	Volatile susp'd solids.	Settleable solids.	Turbidity.
Raw Sewage:								
Run 1	940	44	5.0	1.6	119	73	....	....
" 2	1022	57	4.6	1.5	92	57	2.20	....
" 3	1307	64	4.7	1.8	105	67	2.25	250
" 4	52	34	....	....	96	65	2.25	....
Average	830	50	4.8	1.6	103	65	2.23	250
Miles Effluent:								
Run 1	....	−12	5.3	1.5	48	34	....	....
" 2	....	−92	4.8	1.6	37	28	0.10	....
" 3	....	−53	4.6	1.8	45	33	0.34	161
" 4	....	−99	....	....	32	24	0.21	....
Average	....	−64	4.9	1.6	40	30	0.22	161
Per cent reduction	....	....	0%	0%	61%	54%	90%	36%

(Boulevard Sewer.)

One test, March 25–April 21, 1918, inclusive, 28 days.

Raw sewage	46	126	15.6	....	120	84	1.99	....
Miles effluent	....	−105	11.2	....	40	33	0.19	....
Per cent reduction	....	....	28%	....	66%	61%	90%	....

The effluent was well clarified and showed a good bacterial reduction as indicated by figures cited in Chap. XIV. Most important of all however was the fact that both effluent and sludge were so affected by the acid present as to be stable for considerable periods, so that with a plant of this type no local nuisance need be anticipated.

The fact was however brought out in the New Haven experiments that the effluent from the acid treatment contains so much sulphurous acid and bisulphite that it will take up a very

large amount of oxygen from the water into which it is discharged (Mohlman, 1918). This tendency to absorb oxygen, and to absorb it very rapidly, might be serious in its effect on fish life and perhaps on nitrifying processes, and it would perhaps be necessary, as Dr. Mohlman has suggested to reaerate the effluent in some special way before it is discharged. The effect of the acid upon tanks and other structural elements of the plant also deserves serious consideration.

These minor difficulties could no doubt be overcome if the acid treatment should prove in general efficient and economical. That it is efficient seems reasonably clear; but its economic practicability depends on the value of the grease to be recovered from the sludge. This point will be discussed in detail in Chapter XIII.

Electrolytic Treatment of Sewage. George W. Fuller (1912) has well said that "Electricity for the purification of water or sewage seems to provide wonderful fascination to the lay mind and it is certainly attractive to the scientist." From time to time electrical processes have been vehemently advocated and have enjoyed a transitory vogue, but so far none of them have established themselves as of wide practical usefulness.

Electrical treatment of sewage has usually aimed at two more or less distinct ends, the production of a chemical precipitant for the removal of suspended solids and the formation of disinfectant substances for the destruction of bacteria.

The Webster process, the first to attract general attention, was installed for the experimental treatment of London sewage at Crossness in 1889 and again at Bradford, England, in the following year. The sewage flowed through long troughs in contact with iron electrodes and the principal action was a precipitation of solids by the iron hydrate formed. Webster however recognized also a disinfectant action due to hypochlorites liberated from salts in the sewage itself.

Albert E. Woolf developed this latter phase of electrical treatment by electrolyzing strong brine and treating sewage with the sodium hypochlorite formed. A small plant of this type was constructed at Brewster, N. Y., in 1893.

The general possibilities of electrolytic treatment (of water) were extensively investigated by Fuller in his studies of the purification of Ohio River water at Louisville (see Fuller, 1912,

for a review of this work). He concludes that "as an aid to sedimentation, electrolytic treatment with iron electrodes is capable of practical use. There is no economy in this over the use of salts of iron now on the market."

A plant operating along the general lines of the Webster process was installed at Santa Monica, Cal., in 1908. It proved highly unsatisfactory in operation in its original form, and was materially modified under a new set of patents taken out by the Electro-Sanitation Co. A similar plant has been installed at Oklahoma City, Okla. No careful studies have been reported from either of these plants; but for a time at least both yielded generally satisfactory results, so far as purification was concerned. There can indeed be no doubt that precipitants produced by electrolysis in the plant itself will work just as efficiently and no more efficiently than if manufactured outside. The important question is whether electrolysis in the presence of the sewage is or is not the most economical method of preparing and applying such precipitants or disinfectants as are needed. If the problem is regarded from this practical standpoint and without vague mystical conceptions of the purifying agency of electrical energy as such (which are apt to cloud the matter in the lay mind), a sound conclusion may be reached in any given case by a careful consideration of operating costs; and this conclusion will not generally be in favor of electrolytic treatment.

The most extensive recent studies of this subject have been made at a small experimental plant and later at the 750,000 gallon unit installed at Elmhurst, Long Island, in 1915 (Eng. News, 1916*b*). This plant, known as the Landreth plant, and installed by the Electro-Chemico Corporation of Philadelphia, makes use of simultaneous electrolysis and precipitation by the addition of lime. The sewage flows between sets of parallel steel plates with paddles revolving parallel to the plates. The operation of the small plant was reported on by four groups of experts, and the large unit was studied by Mason, Olsen and Mailloux and by the Bureau of Sewer Plan of the City of New York. The maximum values of current supply were 125 volts and 40 amperes. The time of flow through the electrolytic tanks was 68 seconds and the period of subsequent retention four hours. With 1000 to 1300 pounds of hydrated lime per million gallons the oxygen saturations for raw sewage, machine and tank

effluents were 25 per cent, 32 per cent and 45 per cent respectively, and the bacterial counts, 1,100,000, 846,000 and 800. When lime (1510 pounds per million gallons) alone was used with no current the oxygen saturation values were respectively 28 per cent, 44 per cent and 44 per cent — the counts 210,000, 11,150 and 61,000. Prof. Earle B. Phelps of the U. S. Public Health Service who was called in to review the conflicting reports of the various experts concluded that the final effluent was highly satisfactory, — that there was good basis for honest difference of opinion as to whether the electrolysis contributed materially to the results, — and that the cost was too high to compete under usual conditions with better established processes. The City Board of Consulting Engineers estimated the cost of electro-chemical treatment at $48.19 per million gallons against $39.38 for activated sludge treatment, $36.45 for trickling filters, and $30.63 for straight chemical precipitation; and dismissed the electrolytic process on account of its high cost and uncertainty of results.

Present Status of Chemical Precipitation. Though the chemical treatment of sewage gives an effluent much freer from suspended matter than either the effluent from simple sedimentation or from the septic tank process, and consequently causes less trouble from the clogging of bacterial beds, the amount of sludge produced is very large, over fifty per cent more than the amount which accumulates in a sedimentation tank from the same sewage. This fact, with the cost of the chemicals used, counts heavily against chemical treatment, and in the United States practically no chemical precipitation plants have been built in recent years. Fuller (1909) comments upon the process as follows:

"As a preparatory treatment for filtration its use has much more merit in the case of sewages highly charged with trade wastes than for ordinary domestic sewage. In some European projects chemical precipitation is still held to, because it is believed that its cost is justified by the increased rate at which the filters may be operated. With our dilute American sewage it is not believed that this will be the case under any ordinary circumstances."

There is, however, another side to this question, and there has been a feeling in England that for those cities which can dispose

of sludge by carrying it off in sewage tank steamers to be discharged into the ocean, it would be cheaper and better to remove all the suspended matter by adding chemicals than to allow the suspended matter contained in the effluent from plain sedimentation or septic tank treatment to accumulate upon bacterial beds; and it may be considered as debatable if even for certain inland cities the use of chemicals may not be advisable. The question of chemical treatment as a preliminary process resolves itself into deciding whether or not in a given case the advantages of obtaining an effluent more easily and cheaply treated on bacterial beds than is otherwise obtainable, and with less danger of causing an aerial nuisance, do or do not offset the cost of chemicals and of the disposal of the large amount of sludge formed.

The possible recovery of grease from the sewage sludge is of course a strong argument in favor of such processes of acid precipitation as are embodied in the Miles system described above. This fact, together with the marked disinfectant action which is claimed, makes the plan of treating sewage with sulphur dioxide well worthy of careful study.

CHAPTER VI

PRELIMINARY TREATMENT OF SEWAGE BY THE SEPTIC PROCESS

The Nature of the Septic Process. Septic action may be described as the decomposition, through bacterial agencies, of the sludge in the bottom of a sedimentation tank, with the consequent production of gases and the breaking up and partial liquefaction of the solid matter.

This action takes place whenever sediment or sludge is not frequently removed from a tank. In hot summer weather it begins in a few days after sewage has been allowed to flow through, and may reach its full development in less than a month; while in cold winter weather the bacterial action develops very much more slowly, often taking from two to three months or more. The decomposition of the organic matter is due primarily to anaerobic bacteria, although enzymes as well as worms and other animal forms undoubtedly play an important part.

It has been claimed that not only the suspended matter but also the organic matter in solution is partially decomposed, and, consequently, that the sewage after septic tank action is more easily and quickly oxidized on filter beds by the aerobic bacteria. This is questionable, and the general opinion at the present time is that the advantage of septic tank action lies merely in the reduction of the amount of sludge deposited by sedimentation.

The septic process must therefore be considered as a preliminary method of treatment, similar to sedimentation, the only difference being that in the septic process the sludge is stored in the tank so that a portion of the suspended organic matter may be liquefied with the evolution of gas. As a forcible illustration of this decomposition a description of Dunbar's experiments is well worth noting.

Dunbar says (Dunbar, 1908): " The author has investigated the subject by suspending in septic tanks a large number of solid organic substances, such as cooked vegetables, cabbages, turnips,

potatoes, peas, beans, bread, various forms of cellulose, flesh in the form of the dead bodies of animals, skinned and unskinned, various kinds of fat, bones, cartilage, etc., and has shown that many of these substances are almost completely dissolved in from three to four weeks. They first presented a swollen appearance, and increased in weight. The turnips had holes on the surface, which gradually became deeper. The edges of the cabbage leaves looked as though they had been bitten, and similar signs of decomposition were visible in the case of the other substances. Of the skinned animals, the skeleton alone remained after a short time; with the unskinned animals the process lasted rather longer. At this stage I will only point out that the experiments were so arranged that no portion of the substances could be washed away; their disappearance was therefore due to solution and gasification. The skinned body of a guinea pig was allowed to remain in a septic tank for three weeks, when the clean white bones alone remained. . . . Objects suspended in the sludge itself decomposed almost as quickly as those suspended in the supernatant liquid." Calmette (1909) describes similar experiments in which cooked egg albumin was 99 per cent dissolved in 6 weeks. Of cooked meat 96 per cent disappeared in 6 weeks. Fish meat entirely disappeared in 2 weeks. Cabbage heads were reduced to 1 per cent of their original weight in 6 weeks.

Early Development of the Septic Process. The essential processes of the septic tank are the same processes which have always taken place in the cesspool of our forefathers; for the septic tank is simply a cesspool, regulated and controlled. The comprehension of the fundamental principles involved and the working out of the details of construction and operation best calculated to facilitate the process have gradually developed in the hands of various experimenters.

Leonard Metcalf, M. Am. Soc. C. E., in 1901 presented to the American Society of Civil Engineers a paper entitled " Antecedents of the Septic Tank" (Metcalf, 1901). This paper and the appended discussion describe in an admirable manner the early applications of septic action, and include a number of diagramatic drawings of sewage tanks built previous to 1896.

In 1857 Henry Austin, in a " Report on Means for Deodorizing and Utilizing the Sewage of Towns," described a tank,

designed by him, with a view to separating the solids from sewage before treatment with lime (Fig. 39). This tank closely resembled modern tanks and included, from a constructional standpoint, the essential elements of successful septic tank treatment as practiced to-day.

The Austin tank was built like a septic tank but was not intentionally designed to accomplish what we consider to be the main work of a septic tank to-day, the liquefaction of sludge. The first definitely purposeful attempt to attain this end was

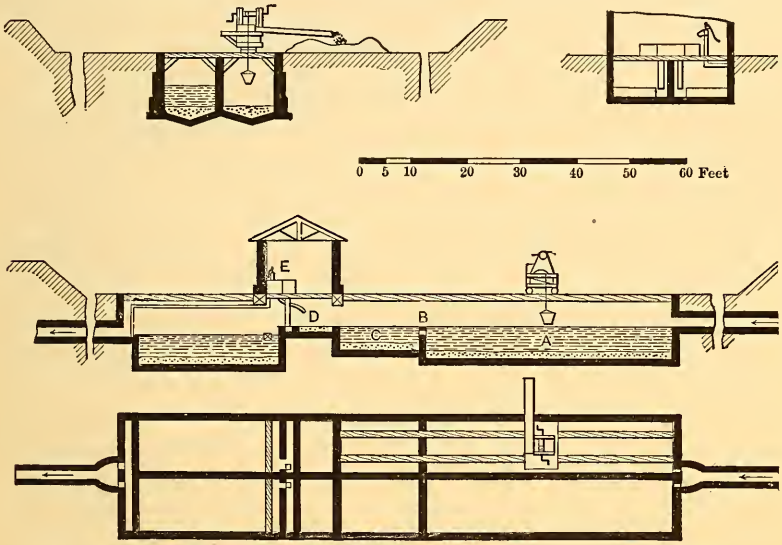

FIG. 39. The Austin Sewage Tank (copied by permission from Metcalf, 1901).

made by Louis H. Mouras of Vesoul, France, who designed the Automatic Scavenger, which in America would be called an overflow cesspool. The Mouras scavenger was " a closed vault with a water seal, which rapidly transforms all the excrementitious matter which it receives into a homogeneous fluid, only slightly turbid, and holding all the solid matters in suspension in the form of scarcely visible filaments." The action was attributed to anaerobic bacteria. This tank was introduced by Mouras about 1860, and was fully described by Abbé Moigno in the *Cosmos les Mondes* in 1881. Automatic scavengers of this type were widely used in Paris because a city ordinance then

forbade the discharge of solid material into the sewers. They were patented in France, England, and the United States in 1881–1882, twenty years after their first use.

Abbé Moigno, speaking of the liquefaction of the solid matter in the Mouras Automatic Scavenger, says:

"Daily observations conducted with a glass laboratory-scavenger have been made, and from these it results that fecal matters introduced on the 29th of August were entirely dissolved on the 16th of September. Even kitchen refuse, onion peelings, etc., which at first floated on the surface, descended after a time to the bottom of the vessel to await decomposition. Everything capable of being dissolved acted in a similar way, and even paper wholly disappeared."

Explaining this action, he says, "May not the unseen agents be those vibrions or anaerobes which, according to Pasteur, are destroyed by oxygen, and only manifest their activity in vessels from which the air is excluded?"

The late Col. George E. Waring, Jr., in his book entitled "The Sanitary Drainage of Houses and Towns," published in 1876, describes a closed tank, built by him at Newport, in which "the solid deposit being organic matter decomposes in the form of ammonia which helps to dissolve the grease and make it soluble, so that both the deposit and the scum are constantly being washed away." In the next decade a number of tanks of this general type were built at various places in the United States. All embodied more or less clearly the principle of storage of sewage in a tank with submerged inlet and outlet. The designs installed at the Worcester (Mass.) Insane Hospital (1876), at Lawrenceville (N. J.) (1882), and the Concord (Mass.) Reformatory (1883), and at Medfield, Mass. (1886), are well shown in Metcalf's figures. In 1883 the late Edw. S. Philbrick described a sewage disposal system designed by him in which there was provided "a tank or tank cesspool in which the solid particles of the sewage may become macerated and finely divided by fermentation before entering the distribution pipes."

Scott-Moncrieff, the well-known sanitary engineer of Ashtead, England, maintained that from a bacteriological standpoint the purification of sewage took place in two stages, and that the first or anaerobic stage should serve as a prelude to further

treatment. In 1891 he built at Ashtead a small plant which consisted of a closed tank filled with stones, for the partial liquefaction of the solid matter in the sewage, and of open trays containing coke, for the second stage, where nitrification was to take place.

In 1893 a plant on the same plan was designed by Scott-Moncrieff for the borough of Towchester. A year later, in 1894, Prof. A. N. Talbot built a sewage tank at Urbana, Ill., in which the liquefying anaerobic action was observed; and a larger plant, with this definite end in view, was designed for Champaign, Ill., in 1895 and built in 1897.

Cameron's Septic Tank. In spite of the pioneer efforts of these earlier workers the anaerobic process of sewage purification owes its practical development chiefly to Donald Cameron

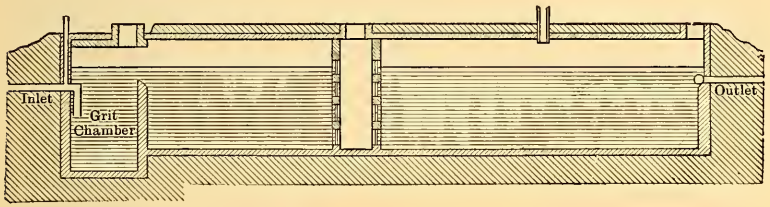

FIG. 40. Cameron's Septic Tank.

of Exeter, England, who holds much the same relation to this process that the Massachusetts State Board of Health holds toward intermittent filtration. In 1895 he installed a water-tight covered basin for the treatment of the sewage of a portion of the city by anaerobic putrefaction and gave it the picturesque name of the "septic tank," by which it has since been known. The tank at Exeter (Fig. 40) was an underground tank of cement concrete, 65 feet long, 19 feet wide, and of an average depth of 7 feet, and having a capacity of 53,000 gallons. The tank was covered with a concrete arch, and a portion near the inlets was made about 3 feet deeper than the rest and partially cut off by a low wall, forming a couple of pockets or grit chambers, to retain sand, grit, and road washings. The inlet was carried down to a depth of 5 feet below the surface, so that air could not make its way down with the sewage, and also so that gases could not escape from the tank back into the sewer. The effluent outlet was also below the level of the liquid, and to avoid currents,

that might be liable to carry floating matter from the surface, it was constructed in the form of a cast-iron pipe carried across the whole width of the tank 15 inches below the surface, on the lower side of which was a continuous opening about half an inch in width. An iron pipe about one and a half inches in diameter extended up from the top of the tank to allow the escape of gases, and the whole tank could be inspected from a central manhole provided with glass windows. In August, 1896, the

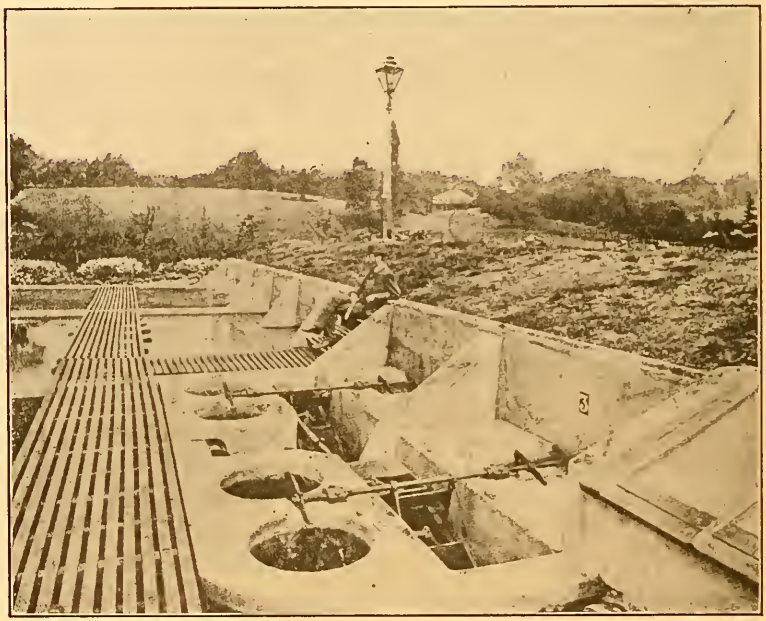

FIG. 41. View of Septic Tank and Contact Beds at Exeter, England.

main sewer of St. Leonard's, a suburb of Exeter, with a population of 1500, and an average daily flow of sewage of 57,000 gallons, was connected with the tank.

The sewage at Exeter flowed slowly through the tank, taking about twenty-four hours in passage. The liquid turned dark-colored, while in the solids collected at the bottom an active fermentation was set up. Bubbles continually rose, carrying with them solid particles, which gathered at the surface to form a scum. Meanwhile the effluent flowing off was freed from gross

floating matter, and its total solids and organic constituents were decreased to one-half and two-thirds their initial value, respectively. The material removed did not, however, merely accumulate in the tank, which was operated for three years without cleaning. At the end of the first year 25 tons of solids had been removed from the sewage, of which it was calculated that 5 tons remained in the tank, and this in the form of a rather stable, peaty deposit, only one-third organic in composition. (Rideal, 1901). These results attracted wide attention all over the world. Cameron's experiments were at once repeated and confirmed at Leeds and Manchester and elsewhere. In many places tanks which had been used for sedimentation and chemical precipitation were turned into septic tanks by merely changing the inlet and outlet so that the sewage could enter the tank, and the effluent leave the tank, 4 to 6 inches below the surface. Remarkable results were reported by some observers, who claimed that all of the organic matter was liquefied so that practically no sludge remained in the tank, and it was believed that at last the sludge disposal problem had been solved. The earlier opinions regarding the liquefaction of sludge by this process have been very much modified, as will be shown later; but anaerobic preliminary treatment has won its place as one of the most valuable processes in the purification of sewage.

Chemistry of the Septic Tank. The most characteristic processes which take place in the septic tank are hydrolyses, of which Rideal (1901) gives the following formula as an ideal type, taking $C_8H_{13}N_2O_3$ to represent the percentage composition of albumin:

$$4\,C_8H_{13}N_2O_3 + 14\,H_2O = 4\,N_2 + 19\,CH_4 + 13\,CO_2 + 2\,H_2.$$

Other similar changes are effected without the addition of water; but all consist essentially in the decomposition of complex organic molecules into simpler compounds, without the addition of oxygen from external sources.

Rideal (1901) discusses the phenomena of anaerobic decomposition very thoroughly and classifies the important processes under eight main heads. 1. The first step is the hydrolysis of the complex albuminous bodies, which again includes two stages, — peptonization, or conversion into a soluble form, and splitting up of the resulting peptones into amino acids, leucin, tyrosin,

etc., together with aromatic bodies. 2. The amino compounds formed under (1) are decomposed further into nitrogen or ammonia and fatty or aromatic acids. Whether nitrogen or ammonia is formed depends on the activity of the septic process and probably upon the types of bacteria present. Both reactions practically occur. In addition to the soluble or gaseous products there is a solid by-product which consists of a dark finely divided sediment, nitrogenous but stable, and resembling the peaty or humus matters in the soil. 3. In the breaking up of the original albuminoid molecule various organic acids are formed which break down to simpler acids and finally to carbonic acid and hydrogen or methane. Formates are decomposed to acid sodium carbonate, carbonic acid and hydrogen. With acetates the process is the same, except that methane takes the place of hydrogen. Lactates may break up in various ways to form propionic acid with valeric or acetic and succinic acids as by-products, or to form butyric acid with propionic acid or hydrogen as by-products. Malates may yield chiefly either propionic, succinic, butyric or lactic acids, with acetic or carbonic acids or hydrogen as by-products. Tartrates may form propionic, butyric or acetic acid, according to the type of fermentation, with alcohol and succinic acids as by-products in the latter case. Citrates produce chiefly acetic acid, and glycerates mainly acetic or formic acid. 4. The urea originally present in the sewage is directly hydrolized to form carbonic acid and ammonia. 5. Cellulose is decomposed to form fatty acids and carbonic acid and either methane or hydrogen, according to the type of micro-organism which is active. Omelianski (1906) records the quantitative results of the action of two of the common types of bacteria present (see Table XXXII), after many weeks of action: Under

TABLE XXXII

END PRODUCTS OF CELLULOSE DECOMPOSITION IN GRAMS

(Omelianski, 1906.)

	Type A.	Type B.
Fatty acids	2.2402	1.0223
Carbonic acid	0.9722	0.8678
Undecomposed residue	0.1272	0.0750
Hydrogen	0.0138	.....
Methane	.....	0.1372

the influence of organisms of Type A the fatty acids consisted mainly of acetic and butyric acids in the ratio 1.7 to 1.0. Under Type B about nine parts of acetic acid were found to one of butyric. 6. Starches, sugars and gums are quickly hydrolized and decomposed to lactic or butyric acids, carbonic acid, hydrogen and water. 7. The decomposition of fats is almost nil under anaerobic conditions. Glycerine is attacked and fats emulsified but the higher acids remain almost unchanged. Under aerobic conditions the fats themselves are acidified by certain species of bacteria, and still more actively by molds (Rahn, 1906). 8. A special set of fermentations liberates the sulphur of the organic molecule in the form of mercaptans or hydrogen sulphide. Most of the sulphur quickly combines with any iron present and is precipitated as finely divided sulphide.

The end results of the whole process are, then: (a) gases, including most of the H, N, CO_2 and CH_4, (b) substances in solution, including most of the ammonia, undecomposed amino bodies and fatty acids, (c) a solid residuum of stable peaty organic matter. The whole reaction is an exothermic one, evolving about 8 per cent as much heat energy as is left in the final products. (Rideal, 1901.)

Comparative Analyses of Influents and Effluents of Septic Tanks. The chief practical result of septic treatment is of course the reduction in suspended solids. Aside from this point, which will be discussed more fully later, the main analytical differences between the influent and the effluent of a septic tank lie in a decrease of albuminoid nitrogen and oxygen consumed and an increase in ammonia nitrogen the latter effect varying in different tanks.

The results of septic treatment in three English and three American cities are brought together in the table on page 146. In comparing them it will be noticed that the effect on ammonia nitrogen varies, this constituent sometimes decreasing appreciably, as at Exeter, but generally remaining fairly constant. Sometimes, as at Worcester, it exhibits a marked increase. The reactions in the septic tank naturally vary materially with the original composition and age of the sewage. In a very fresh sewage there is always a considerable formation of ammonia nitrogen by the decomposition of more complex organic bodies. If this process has been completed when the sewage is subjected

TABLE XXXIII

ANALYTICAL RESULTS OF SEPTIC TREATMENT

Place.	Material.	Solids.		Nitrogen as —		Oxygen consumed.	Remarks.
		Total.	Suspended.	Ammonia N.	Albuminoid N.	Total.	
Exeter	Sewage	778	350	44.4	.....	29	April to June, 1897.
	Tank effluent	593	154	32.5	.....	20.1	
Leeds	Sewage	1690	622	24.7	11.7	127	Feb., 1899, to Jan. 15, 1900.
	Tank effluent	1090	183	21	5.2	58.5	
Birmingham	Sewage	1967	676	31.9	13.7	153	Septic tank, No. 1, 1901, open tank.
	Tank effluent	1399	245	43.3	18.7	108	
Lawrence	Sewage	769	232	38.1	7	49.5	Tank A, January, 1898, to January, 1903.
	Tank effluent	597	107	37.7	3.3	27.3	
Boston	Sewage	.....	.....	18	5.8	42.3	Average of 6 tanks, 1903–1905.
	Tank effluent	.....	.....	20.6	3.6	36.7	
Worcester	Sewage	832	311	20.2	8.4	110.7	1902, weekly samples.
	Tank effluent	625	214	25.8	5.6	70.7	

Note. — Oxygen consumed in four hours at 80° F., in the English results: in two minutes boiling, at Lawrence, in ten minutes, at Worcester. Solids in Lawrence figures for year 1902 only.

to septic treatment a decrease in ammonia nitrogen may be expected in the tank. Albuminoid nitrogen and oxygen consumed in each case fall to one-half or two-thirds of their initial value. The evidence accumulated by the Royal Sewage Commission indicated that an increase of ammonia nitrogen was the general rule in English septic tanks, while the albuminoid nitrogen was reduced 38 to 54 per cent at Exeter, 50 per cent at Leicester, and 36 per cent at Birmingham. The oxygen consumed was reduced 25 to 33 per cent at Exeter, 50 per cent at Accrington, 50 per cent at Leeds, 36 to 60 per cent at Leicester, and 29 per cent at Birmingham. (Martin, 1905.)

Gas Production in the Septic Tank. The gas produced in the septic tank is a close measure of the amount of organic decomposition. Calmette (1909) calculates that a liter of methane represents either 1.7 grams of albumin or 2.4 grams of cellulose. The volume of gas was found by Fowler at Manchester (1901) to be 7.5 gallons per 100 gallons of sewage, and Clark (1900) at Lawrence obtained concordant results. Kinnicutt and Eddy (1902), on the other hand, found as an average, only about half this amount (3.9 gallons) produced by the septic treatment of the acid-iron sewage of Worcester. The composition of the gas also varies. The table below has been prepared from data given by Rideal (1901) for Exeter, from Kinnicutt and Eddy's analyses of the gases from the experimental septic tank at Worcester, and from analyses of the gas from the Lawrence tank by A. H. Gill. The Worcester results represent weekly analyses for a year.

At Worcester, during the warm summer months the amount of methane rose to 81 per cent and the carbon dioxide to 8.85, the nitrogen falling to 8 per cent; and careful examinations of

TABLE XXXIV

COMPOSITION OF SEPTIC TANK GASES

Per cent of various constituents.

	Methane.	Nitrogen.	Carbon dioxide.	Hydrogen.	Other gases.
Exeter	20.3	61.2	0.3	18.2	.....
Worcester	75.2	17.4	5.9	0.3	1.4
Lawrence	78.9	16.3	3.4	.....	.....

large volumes of gas failed to show the presence of either hydrogen sulphide or carbon monoxide.

Bacteriology of the Septic Tank. Our knowledge of the bacteriology of the septic tank is somewhat fragmentary and unsatisfactory. What little is known of organic decomposition is well reviewed in Lafar's "Technical Mycology" by Miquel (1906), Hahn and Spieckermann (1906) and Omelianski (1906). According to Bienstock (1899), the initial decomposition of native proteins into aromatic oxyacids, etc., depends upon a group of obligate anaerobes of which his *B. putrificus* is a type. Rettger (1908) and others have confirmed these results. On the other hand, it is strange that obligate anaerobes have never been found in any numbers either in feces (Bienstock, Rettger) or in sewage (Winslow and Belcher, 1904). Possibly symbiotic phenomena play an important part here; or the active organisms may be of types which cannot be cultivated on ordinary media. The later changes leading to the formation of hydrogen, carbon dioxide, methane and nitrogen are carried forward by a great many types of metatrophic bacteria, which can live either in the presence or the absence of oxygen. The simpler carbohydrates and organic acids are broken up by numerous species, *B. coli*, for example. The ammoniacal fermentation of urea may be due to various cocci and bacilli (Miquel, 1906). The two types of cellulose fermentation are carried out by specific anaerobic spore-forming rods (Omelianski, 1906). The sulphur fermentations have their own peculiar groups of organisms.

The bacteria, of course, effect all these changes by the secretion of enzymes, either within or without the bacterial cell. Much of the effect may be due to enzymes discharged from the cell and distributed through the liquid contents of the tank.

Effect of Temperature upon the Septic Process. One of the most striking characteristics of the septic process is the effect of temperature thereon. The following data, compiled by Fuller from Eddy and Kinnicutt's results, show the greatly increased activity of the experimental septic tank at Worcester during the warmer months:

RATIO OF THE VOLUME OF GAS PRODUCED EACH MONTH AT WORCESTER TO THE ANNUAL MEAN

January	30	July	140
February	62	August	167
March	48	September	170
April	51	October	116
May	100	November	115
June	148	December	65

The close relation between gas production and temperature is also shown in Fig. 42, plotted, from the same Worcester data tabulated above, by Winslow and Phelps (1906). As a result of

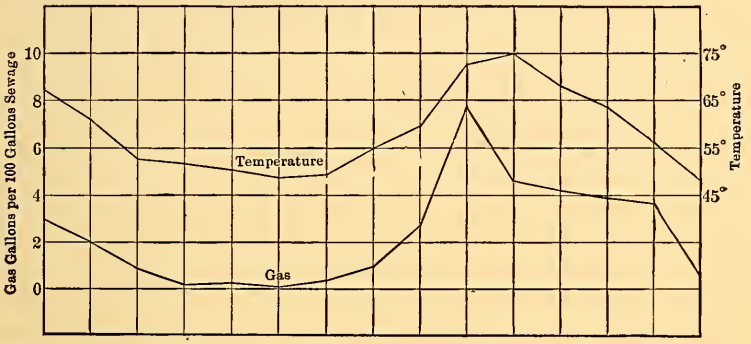

FIG. 42. Gas Production and Temperature, Worcester Septic Tank.

the temperature curve of septic action there is a great difference in the amount of sludge decomposed during different portions of the year. In winter, sludge gradually accumulates, while in summer, liquefaction exceeds deposition. Hence observations of the efficiency of a tank should cover all seasons of the year. It is important to remember that the periods of active fermentation are often marked by serious deterioration in the quality of the septic effluent on account of the fact that the gas bubbles rising from the boiling sludge stir up the finer suspended material and carry it over to the outlet. The temperature factor has an important influence in controlling the varying success of the septic tank in different countries. An almost complete absence of liquefaction was reported in Russia from the results of Dzerszgowski's tank at Tsarskoé-Sélo. In India, on the other hand, septic tanks operate with marked success

(Clemesha, 1910). A tank at Matunga was only emptied 3 times in 8 years and the volatile solids were reduced from 86 per cent in fresh sludge to 28 per cent in septic sludge (Calmette, 1909).

The course of septic action with a given sewage may frequently be profoundly modified by peculiarities in the composition of the sewage itself. Thus the Worcester sewage, which contains 100 parts per million of free acidity, reckoned in terms of sulphuric acid, and 50–80 parts of iron in solution, yields, as has been pointed out, only 3.9 gallons of gas per 100 gallons of sewage, while results at Manchester and Lawrence show nearly double

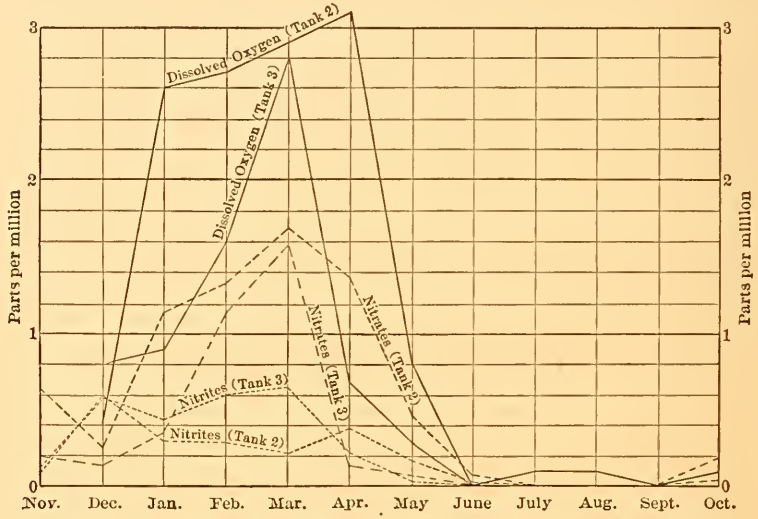

FIG. 43. Seasonal Variations in Septic Tank Effluent at Waterbury (Taylor, 1909*a*).

this amount. Furthermore the precipitation of iron sulphide has led to the presence of considerable fine suspended matter in the Worcester effluent (Kinnicutt and Eddy, 1903).

An interesting study of the delayed ripening of a septic tank has been made at Waterbury, Conn. (Taylor, 1909*a*). Septic tanks started in September, 1905, showed no evidence whatever of septic action until May, 1906. After the latter date active ebullition set in and oxygen disappeared from the septic effluent. The delayed onset of septic action is well shown by Taylor's curve (Fig. 43). There was too much oxygen in the applied

sewage for the inception of anaerobic changes on any appreciable scale during the colder months of the year.

It may well be that in some cases inadequate septic action is due to the absence of specific types of bacteria essential to the process, rather than to chemical peculiarities in the composition of the sewage. We greatly need more definite scientific knowledge of the bacteriology of septic action. Only when such knowledge as this has been acquired can the process be rationally controlled.

The Capacity of Septic Tanks. A septic tank is of course essentially a sedimentation tank, with modifications, and the same broad rules of construction apply to both. What has been said of tanks in general in Chapter IV need not therefore be repeated here. The chief differences between the Cameron septic tank and the ordinary sedimentation tank lay in the fact that the former was covered, that it had a submerged inlet and outlet, that the flow was slower and, most important of all, that the sludge was allowed to remain in the tank to be decomposed, instead of being removed at frequent intervals. The first of these characteristics has now been generally abandoned except in cold climates and the last is a difference in operation only. The larger size of tanks and the arrangement of the inlet and outlet remain as the only peculiarities of construction which distinguish the septic tank from tanks of other forms; and indeed the inlet and outlet of a tank for plain sedimentation are, in practice, usually submerged, as in the septic tank.

The septic tank is always made relatively larger than the plain sedimentation tank, partly in order to allow for solution of fine suspended matter and partly in order to provide space for large accumulations of sludge without unduly increasing the rate of flow. This extra capacity may sometimes be provided for in the form of an additional unit which can be thrown in and out of service as desired. The lower limit of capacity is of course set by the fact that the period of storage must at least be long enough for the settling out of the maximum quantity of suspended solids. From laboratory experiments on sedimentation it appears that this maximum is about 80 per cent and that it will be nearly reached by six hours' storage (Steurnagel, 1904). In practice it has often been found that a longer period is of advantage, perhaps for the liquefaction of suspended solids of too fine a char-

acter to settle out. The Leeds results in the table below indicate an appreciably greater removal in twenty-four hours than in twelve hours, while further prolonging the period to forty-eight or seventy-two hours is of no advantage.

TABLE XXXV

AVERAGE OF ANALYSES ILLUSTRATING THE EFFECT OF DIFFERENT RATES OF FLOW THROUGH OPEN SEPTIC TANKS

(Leeds, 1905.)

	12 hours' flow.		24 hours' flow.		48 hours' flow.		72 hours' flow.	
	Parts per million.	Purification (per cent).	Parts per million.	Purification (per cent).	Parts per million.	Purification (per cent).	Parts per million.	Purification (per cent).
Total solids.....	1250	....	1110	....	1120	....	1050	....
Suspended solids	272	52	162	71	155	73	141	76
Nitrogen as —								
Ammonia N..	18.2	22	17.5	24	18.8	19	20.8	37
Albuminoid N	6.3	50	5.2	58	4.5	64	4	52
Oxygen consumed in 4 hours at 80° F.	74.2	45	68.8	49	61.2	55	51.1	55

The Columbus experiments (Johnson, 1905) showed that the most economical period of subsidence was longest when a strong sewage was being treated. With sewage diluted with storm (surface) water the suspended matter will, relatively speaking, settle out more quickly on account of its mineral character. With dilute night sewage, the rate of subsidence of the suspended matter is slow; but on account of the comparatively small initial quantity of suspended matter in such sewage, the amount remaining, after a given period, may be less than the amount remaining in a stronger sewage subjected to the same sedimentation. As a result of the Columbus experiments, there was recommended a sedimentation tank holding 8 hours' flow of sewage, to be operated on the septic plan.

Valuable experiments upon the "tanking" of sewage, with reference to the determination of proper shape and capacity, have been made by F. Wallis Stoddart (1905) at the Horfield sewage works at Bristol, England. The result of these experiments was to show that, where the flow is subject to variations similar to those which occurred at Bristol, the tank should be

designed for a 12-hour period, based on the average dry-weather flow. In this way the normal increase of about 50 per cent, which occurs during certain portions of the day, will change the rate of flow to correspond with the most favorable period (*i.e.*, 8 hours), and in times of rain the tank will still be able to handle the sewage which comes to it.

The Royal Commission on Sewage Disposal makes the following statement in regard to the proper capacity for septic tanks:

"The rate of flow through a septic tank is consequently a matter in which the needs of each place require to be taken into account; but from general experience and a consideration of the evidence, we think it may safely be said that at few places should the sewage remain more than 24 hours or less than 12 in a septic tank."

Alvord (1902) and other American engineers provide shorter periods, often only four to eight hours, and some tanks operated on this principle, like that at Lake Forest, seem to work well. On the other hand, short periods of septic treatment at Wauwatosa and East Cleveland have yielded less satisfactory results (Winslow, 1905).

The following table gives the capacities of representative tanks, mostly operated on the septic plan, in use in Ohio in 1908. Since that date the entire plant at East Cleveland has been abandoned, and the tanks at Lakewood and Mansfield have been remodeled as two-story tanks.

TABLE XXXVI

OPERATING DATA FOR OHIO SEPTIC TANKS

Place.	Actual capacity of tank, gallons.	Average linear velocity mm. per sec.	Hours flow.	
			Average.	Minimum.
Delaware	100,000	0.75	5.5	2.4
East Cleveland	170,000	0.68	10.6	.....
Geneva	39,000	0.74	5.2	2.1
Kenton	18,800	0.14	19	1.2
Lakewood	300,000	0.50	12.4	1.8
London	34,700	0.15	16.6	.....
Mansfield	1,000,000	0.32	24	.....
Marion	414,000	0.49	15	5
Westerville	22,000	0.15	14.6	2.6
Sandusky — Soldiers' and Sailors' Home	109,000	0.22	15	8.5

It is important that the septic period should not be too prolonged, since the anaerobic fermentation, if carried too far, may produce an effluent difficult to nitrify. Furthermore, it is probable that even the liquefying action itself may be checked by excessive concentration of waste products. An experiment at Lawrence is suggestive. A small septic tank was dosed, not with sewage, but with the more concentrated sludge from settled sewage. For six months the storage period was from five to fifteen days and sludge accumulated, filling up 60 per cent of the tank. The rate was then increased, so that the storage period was reduced to forty-nine hours, when the accumulated sludge decreased to 8 per cent and did not further increase for a year (Massachusetts, 1901). At Leeds it was found that a seventy-two-hour septic period interfered with the solution of sludge (Leeds, 1905). Clark and Gage (1905) have shown that certain types of bacteria especially active in sewage purification increase during the first twenty-four hours of septic treatment and then fall to numbers smaller than are present in raw sewage. It seems possible that too long a period of action may thus actually favor the accumulation of sludge while producing an effluent hard to nitrify. Alvord (1902) for these reasons suggests the use of an "elastic tank" with separate compartments, which can be included in or thrown out of the system to adjust it to varying conditions of flow and temperature.

Working on the same assumption, — that excessive septic action was to be avoided, — Winslow and Phelps (1911) suggested the use of a deep tank built on the Dortmund pattern with the sewage influent at the bottom so that the sludge would be continually washed in a current of fresh sewage. This device, called a Biolytic Tank, operated very successfully with Boston sewage, 72 per cent of the deposited solids being liquefied. At Chicago on the other hand Langdon Pearse has obtained unsatisfactory results with a tank of this type.

The age of the sewage on reaching the plant is of course an important factor in designing a septic tank. A small plant treating fresh and undecomposed sewage, particularly if, as is usually the case, the hourly variations of flow are great, calls for a relatively large tank. One day's flow might be considered a minimum capacity for such a plant, and two days' flow would probably prove more satisfactory.

Other Details of Septic Tank Construction. The second important point in septic tank construction is the provision of submerged inlets and outlets, so arranged that neither sludge nor scum may be unduly disturbed. The outlet of the original Cameron tank, as noted above, was a pipe running across the entire end of the tank 15 inches below the surface, with a slit in its under side. The outlets from the septic tanks at Mansfield, Ohio, were in the form of 98 2-inch pipes arranged in two rows at depths of 2 feet and 2 feet 6 inches below the flow line. At Saratoga, N. Y., each tank was provided with two horizontal rows of similar 2-inch outlet pipes about 3.5 feet below the flow line.

An important though inexpensive feature of the design of all sedimentation tanks are the baffle walls and suspended baffles, or scum-boards, to prevent the forward movement of the deposited matter as well as the floating matter or scum, and, in addition, to cause a more thorough displacement as the sewage passes through the tank.

Scum-boards may be attached to the walls in such a manner that they are free to move vertically, and hence will rise and fall with the sewage. Instead of boards, iron plates, rigidly attached to the walls, are sometimes used. As both iron and wood, when used for this purpose, deteriorate more or less rapidly, it may be better to build the baffles of reinforced concrete in the form of deep, narrow beams, reaching across the tank.

The covering of septic tanks, although recommended by Cameron, has been found quite unnecessary for the maintenance of anaerobic conditions. If sewage be merely allowed to run slowly through an open tank, oxygen is generally used up much faster than it can be absorbed from the surface. At Manchester the results from closed and open tanks under like conditions showed no marked difference, and in similar experiments at Leeds the open tank gave slightly better results, as shown in Table XXXVII. For promoting anaerobic conditions tight covers are therefore needless.

The question of whether or not to cover a tank must be decided mainly in relation to its location with reference to habitations. If the tank is to be situated (say in the case of small plants) within 500 feet of dwellings, or if it is to be near a public highway or park, it would be safest to provide a roof, to control

TABLE XXXVII

EFFICIENCY OF CLOSED AND OPEN SEPTIC TANKS AT LEEDS, ENGLAND

(Harrison, 1900)

	Parts per million.				
	Solids.		Nitrogen as —		
	Total.	Suspended.	Ammonia N.	Albuminoid N.	Oxygen consumed in 4 hours at 80° F.
Open tank:					
Crude sewage.....	1710	633	23.6	11.3	124
Effluent.....	1110	172	20.6	4.9	54.3
Closed tank:					
Crude sewage.....	1720	666	25.5	12.4	131
Effluent.....	1130	197	20	5	69.3

odors emanating from the tank. Where the sewage is of such a nature that its retention in the tank causes the formation of a scum, it is probable that this scum, if exposed to the weather, will produce odors, especially after it has been wet and moisture is evaporating from its surface; and the mechanical disturbance of the scum caused by rain or wind will add to the odors.

Another point which should be considered in connection with providing a roof, is the winter temperature of the sewage taken in connection with the latitude of the place in which the tank is located. Where the water supply of the city is from surface sources the temperature of the sewage is likely to be very near the freezing point; but where the sewage is made up largely of ground water, the temperature will be somewhat higher. Thus, at Saratoga, N. Y., having a surface water supply, the sewage in the covered tanks has been known to freeze, whereas the sewage at places in Ohio, having ground water supplies, has not frozen even when exposed in uncovered tanks.

In this connection it might be mentioned that roofs over tanks in which septic action is taking place should always be suitably ventilated in order to prevent explosive gases accumulating under them. A serious explosion occurred at the Saratoga Springs plant on Jan. 26, 1906, the entire roof of a 51.5 x 91.5 ft. basin being lifted off and ruined. It was believed that a burning match had been dropped through a manhole by one of the laborers employed at the plant. Other explosions have been

reported at Florence, N. C., and from Cromer, Exeter, Ilford and Sherringham in England (Metcalf and Eddy, 1916).

As to the kind of roof, there may be used either a wooden pitch roof or " housing," which should be high enough to permit ready access to, and inspection of, the contents of the tank, or a con-

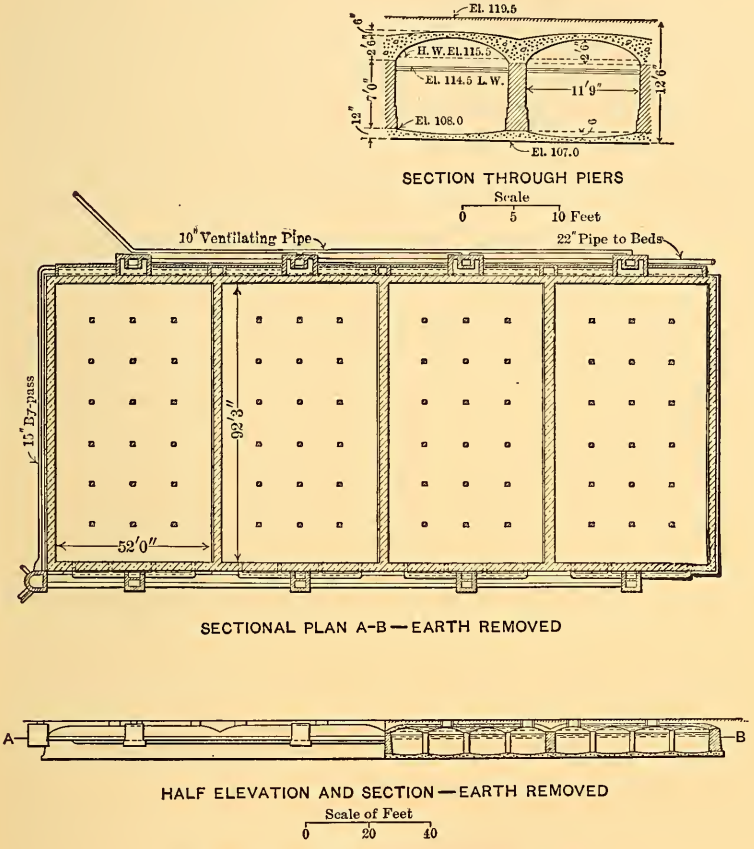

Fig. 44. Septic Tank at Mansfield, Ohio.

crete roof provided with a sufficient number of manholes of generous size, to make the interior of the tank accessible and easily inspected. It would be desirable to have these manholes not more than 15 to 20 feet apart. Concrete roofs may be composed of reinforced beams and slabs, or they may be in the form of groined arches. The latter type is shown in Fig. 44, which

represents the tank at Mansfield, Ohio. It is the usual custom to cover the groined arch roof with one or two feet of earth, and in some cases this is done where the slab construction is used.

Septic tanks containing filling material of various kinds have been used in certain cases for increasing the removal of suspended solids by surface contacts. Scott-Moncrieff in his Ashtead experiments used what was really a septic tank filled with stone. This principle has been applied in many other cases to the construction of anaerobic filters, lateral filters, etc., of various types. The so-called " ladder filters " tested at Leeds, formed by a series of trays of stone, from one to the other of which the sewage flowed continuously, operated on this principle, and worked very badly. At Salford, roughing filters containing one-fourth inch to two-inch gravel were used for continuous filtration at a rate of 20 million gallons per acre per day. It was intended to wash these filters by upward flow with artificial aeration, but they have clogged seriously (Baker, 1904). At Lawrence a thorough study has been made of various strainers which operate with more or less continuous flow. All such devices, as well as the anaerobic filters installed at certain sewage plants in the Middle West, act like septic tanks, with the additional straining action due to the included material. Against this increased straining action must be set the tendency to clog and the difficulty of cleaning.

At many plants special devices are used for aerating the tank effluent. The purpose of this is to remove gases and products of bacterial decomposition, and if possible to cause the effluent to absorb oxygen, with the idea of making it more readily nitrifiable in the filters. It is debatable whether with effluents of ordinary concentration, such aeration is necessary. There is a distinct disadvantage, due to the liability of causing unnecessary odors, and there is also a loss in temperature in winter weather. On the other hand, with strong effluents having no dissolved oxygen, the process of aeration may perhaps be desirable.

In a few places, notably at Baltimore, Md., the sludge from septic tanks is pumped to special sludge digestion tanks where it is allowed to undergo intensive anaerobic decomposition. This process might seem a logical outcome of the tendency to separate the two processes of sedimentation and sludge digestion, exemplified in the Imhoff tank which will be discussed in the following chapter. Actual records of the performance of

such sludge digestion tanks are, however, too meager to permit of a conclusion as to the real value of such a procedure.

The Saratoga Tank. The tank at Saratoga, N. Y., built by F. A. Barbour, may be taken as one of the best types of successful septic tank construction in the United States (Barbour, 1905). The town is a famous pleasure resort with a normal population of 12,000, increased to 50,000 in midsummer. The daily flow of sewage varies correspondingly from 1,250,000 gallons a day to double this amount. The discharge of the crude sewage into brooks led to serious nuisances, and the town was forced to pay over $20,000 in damages; so in 1903 septic tanks and sand filters were installed. " The sewage, after screening, is pumped to four septic tanks, each 91.5 feet long by 51.5 feet wide, having a total capacity of 1,000,000 gallons. The depth of sewage is 7.75 feet at the inlet and 8.25 feet at the outlet end.

" The entire structure is built of Portland cement concrete. The outside walls are 2 feet thick at the springing line of arches, vertical on the inside and with a batter of about $1\frac{1}{3}$ inches per foot on the outside. The division walls are 2 feet thick at the springing line and 3 feet thick at the level of the underside of floor. The piers are 18 inches square, the head being enlarged to 22 inches and the footing to 30 inches.

" The roof is of elliptical groined arch construction, the span being 11 feet 6 inches and the rise 2 feet 6 inches. The thickness at crown is 6 inches and the plane of extrados is depressed 9 inches over the piers. This depression is drained by a 2-inch pipe through the roof into the tanks.

" The floor is of inverted spherical groined arch construction, 6 inches thick at the center and 12 inches thick at the piers.

" The force main ends in a chamber, from which a pipe leads across the inlet ends of the tanks. This pipe is carried by a concrete bracket reinforced by old railroad iron. Inlet chambers permit the shutting off of one or more tanks as desired. A by-pass pipe leads from the chamber at the end of the force main around the tanks, so that raw sewage can be applied directly to the beds. Inside of the tanks the inlet pipe is split and carried across the end of tank on a concrete bracket, four openings being provided for the discharge of the sewage at an elevation 3.5 feet below the high-water line.

" The septic effluent escapes from the tanks through two hori-

zontal rows of 2-inch pipes — ninety-six in all, set at an elevation about 3.5 feet below high-water line — into a narrow chamber extending the entire width of tank, from which it flows over a weir into the outlet chambers and thence to beds" (Barbour, 1905). A 24-inch sludge gate permits the emptying of the sludge onto the sludge beds located directly in front of the tanks, and 12-inch gates at a higher elevation make it possible to draw off

FIG. 45. View of Interior of Septic Tank at Saratoga, N. Y. (courtesy of F. A. Barbour).

the clear liquid between the scum and deposit and apply it to any beds previous to the discharge of the sludge. The cost of the septic tanks was $15,500.

A special device, shown in Fig. 46, was installed, for aerating the septic effluent before it passes to the sand beds. The effluent flows over perforated sheet-iron plates, hung in three layers around a central riser pipe. The sewage entering the septic tank contained, in certain tests reported by Barbour, 4.3 per cent of the oxygen necessary for saturation. The septic effluent contained none; but after aeration the value rose to 70.4 per cent,

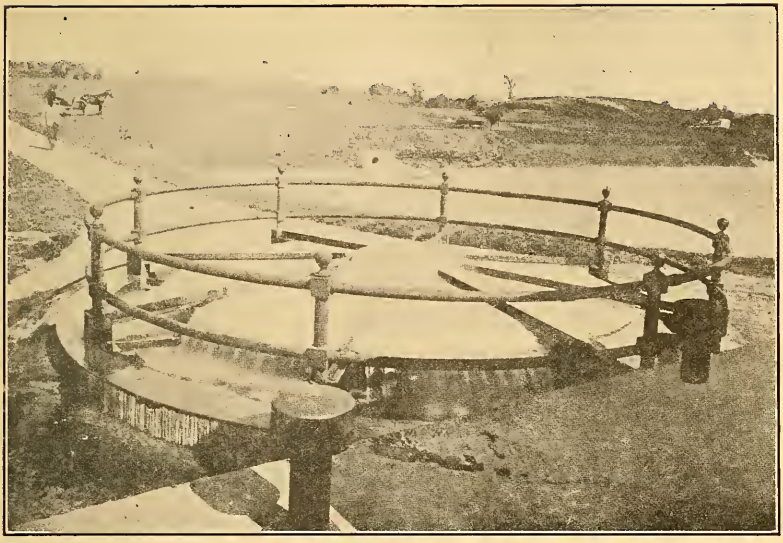

FIG. 46. Sewage Aerator at Saratoga, N. Y. (courtesy of F. A. Barbour).

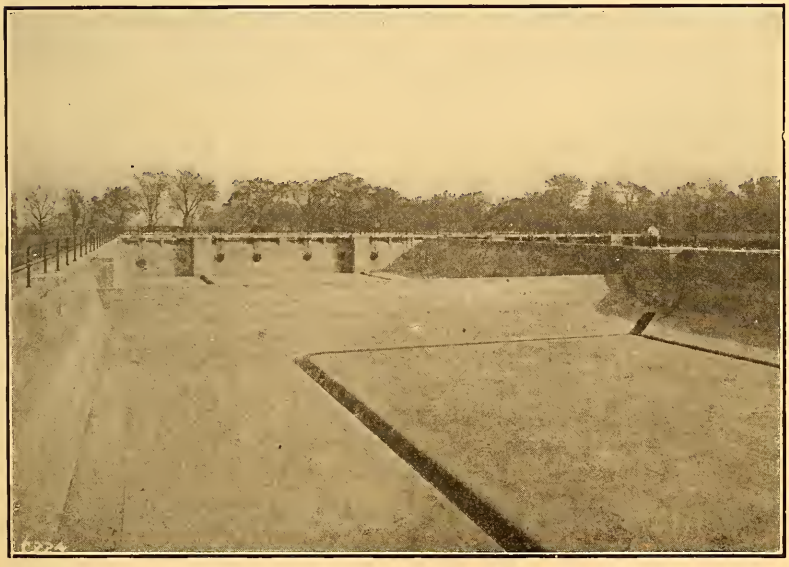

FIG. 47. Open Septic Tank at Columbus, Ohio (courtesy of J. H. Gregory)

falling again to 40.4 per cent before the effluent reached the beds. The rapid decrease after aeration shows the avidity for oxygen of the organic matter present and indicates that the process must materially facilitate the later work of nitrification.

An excellent example of open septic tank construction is shown in Fig. 47 which represents one of the original secondary tanks at Columbus, Ohio. The character of the walls and bottom baffles, the inlet pipes and sludge drains are all well illustrated. The Columbus tanks have since been remodeled as Imhoff or two-story tanks as described in the next chapter.

Practical Results of Septic Treatment. The first point to consider in judging the efficiency of septic treatment is the reduction of suspended solids, measured by direct comparison of the applied sewage and the septic effluent. Tanks at Exeter, Leeds and Birmingham, as noted in the table on page 146, show a reduction of 56, 71 and 64 per cent, respectively. At Leicester the removal has ranged from 60 to 70 per cent. Experiments on London sewage at the Crossness outfall showed that six hours' sedimentation, in what was really a septic tank, gave a reduction from 281 to 125 parts of suspended solids (November, 1900, to March, 1901), and in another series (March to October, 1901) from 253 to 143 parts, a removal of 56 and 43 per cent, respectively. At Leeds 127 parts appeared in the septic effluent from March to June, 1899, 156 parts from July to October and 213 parts from November, 1899, to February, 1900 (Leeds, 1900). At Huddersfield the septic effluent contained 66 parts in August, 1900, 82 parts in September, 113 parts in October, 122 parts in November and 117 parts in December (R. S. C., 1902). These results illustrate the gradual increase in suspended solids discharged in the effluent, as the accumulation of sludge increases.

The table on page 163, from the final report of the British Royal Commission (R. S. C., 1908), gives a good idea of the results attained at a number of representative English plants.

Septic tanks in the United States show reductions of 65 per cent at Saratoga (Barbour, 1905), 61 per cent at Lawrence, Mass. (Mass., 1904), 40 per cent at Boston (Winslow and Phelps, 1907), and 35 per cent at Worcester (Kinnicutt and Eddy, 1903).

Fuller (1912) cites figures showing a removal of 61–63 per cent of suspended solids at Plainfield, and 59–65 per cent at Columbus.

TABLE XXXVIII

Name of place.	Character of sewage.	Time of passage through tanks.	Per cent average reduction in suspended matter.	Suspended solids in tank liquor, parts per million.	Remarks.
Accrington	Strong domestic sewage	Once in 42 hours	50	194	
Andover	Domestic, containing some brewery refuse.	Once in 19.3 hrs. (dry-weather flow); in 13.1 hrs. (max. flow).	30	110	8 mos. after incomplete sludging.
Caterham	Exceptionally strong domestic	Apparently 48 hrs. flow, when 3 out of the 4 hourly sets were drawn.	47	222	1 mo. after sludging.
Exeter (Main Works)	Strong domestic	Once in 11.5 hours (dry-weather flow).	66	125	18 mos. after sludging.
Exeter (St. Leonards)	Weak domestic	Once in 17.8 hours (dry-weather flow). Once in 8.6 hours (max. flow).	68	84	12 mos. after sludging.
Hartley Wintney	Domestic, with a considerable proportion of brewery refuse.	Once in 31.7 hours (dry-weather flow).	54	151	After 3½ years without sludging.
Guildford	Strong domestic, containing a large proportion of brewery refuse.	Once in 36.7 hours	.....	159	Tank full of sludge.
Knowle	Domestic	Once in 15.2 hours (dry-weather flow). Once in 3.2 hours (max. flow).	57	84	
Prestolee	Weak slop water sewage	Once in 22 hours (dry-weather flow).	42	32	After 4½ years without sludging.
Rochdale	Strong manufacturing sewage, containing large quantity of wool scourings, etc.	Once in 30 hours	86	53	Tank in use for only 3 mos.
Slaithwaite	Dilute domestic (mainly slop-water).	Once in 13.6 hours (dry-weather flow).	34	71	After 5½ years without sludging.
York	Weak domestic	Once in 26 hours	75	53	Two tanks, one in use 18 months and the other 6 months.

These are all average results. Many septic tanks in warm periods, when fermentation is active and gas bubbles are rising in great numbers, discharge a considerably greater amount of suspended solids than at other times. When gas ebullition is unusually violent, more suspended matter may leave the tank than enters it. This is strikingly illustrated by the table below showing results of examinations of sewage tanks in Ohio (1908). It will be noted that most of the observations tabulated were made during the warmer months.

TABLE XXXIX

REMOVAL OF SUSPENDED SOLIDS IN OHIO SEPTIC TANKS

Place.	Date, 1906–'07.	Rate of sewage flow,* gals. in 24 hours.	Hours, flow.	Suspended matter.		
				Parts per million.		Percentage removal.
				Crude sewage.	Septic sewage.	
Ashland	July 11–12, '06	150,000	6.3	70	50	29
Ashland	Apr. 24–25, '07	375,000	2.5	50	70	−40
East Cleveland	June 26–27, '06	365,000	11.2	210	400	−91
East Cleveland	July 11–12, '07	390,000	10.4	110	180	−64
Geneva	June 21–22, '06	181,000	5.2	60	65	− 8
Geneva	June 24–25, '07	204,000	4.6	90	150	−67
Kenton, N. District	October 5, '06	18,000	24	220	180	18
Kenton, N. District	July 2–3, '07	17,000	27	70	120	−72
Lakewood	June 12–14, '06	395,000	18	45	45	0
Lakewood	June 12–14, '07	1,150,000	6.2	100	140	−40
Mansfield	May 28–29, '07	1,058,000	23	75	110	−47
Marion	May 23–25, '06	415,000	24	150	85	43
Marion	Nov. 8–9, '06	370,000	27	90	140	−55
Marion	Apr. 9–11, '07	578,000	17	40	45	−12
Sandusky, Soldiers' and Sailors' Home	June 5–6, '06	155,000	17	95	90	5
Sandusky, Soldiers' and Sailors' Home	May 8–9, '07	174,000	15	95	90	5

NOTE. Suspended matter to nearest 5 parts below 100 and to nearest 10 parts above 100.

It is important to note that the sludge remaining in the septic tank is of a different character and is often much less offensive in nature than the sludge deposited from fresh sewage. Calmette (1909) found the differences between fresh sewage sludge and septic sludge to be quite marked, as indicated in Table XL:

TABLE XL

PERCENTAGE COMPOSITION OF FRESH AND SEPTIC SLUDGE

	Fresh sludge.	Septic sludge.
Volatile solids	45.8	32.6
Fixed solids	54.2	67.4
Nitrogen	2	1.3
Carbon	27.9	19.5
Fats	15.8	8

The action of the septic tank on dissolved solids is a variable one, as shown in the table below, compiled by Kinnicutt, with the addition of figures from one of the Lawrence reports:

TABLE XLI

REMOVAL OF SOLIDS BY THE SEPTIC TANK

(Kinnicutt, 1902; Clark, 1904)

Place.	Solids removed (per cent of total).	
	Dissolved.	Suspended.
Exeter	−2.57	56.01
Lawrence	2.12	61.60
Leeds	12.05	70.37
Manchester	15.45	57.06
Worcester	20.67	25.57

It will be noticed that a slight removal of dissolved solids occurs, except at Exeter, reaching a considerable amount at Worcester. The phenomena in the case of Worcester are peculiar, on account of the acids and iron salts in the sewage. In the first place, all the reactions are hindered by the antiseptic action of these substances. The reduction of albuminoid nitrogen is small, only 20 to 25 per cent; the gas production is only half that at Lawrence and Manchester; and the liquefaction of sludge is imperfect. In the second place, the proportionate decrease of suspended solids is small and that of dissolved solids great, on account of the reduction and precipitation of iron as sulphide.

In addition to the reduction in suspended solids, it is claimed

by some experts that the anaerobic putrefaction brings the soluble constituents into a form in which they are more easily acted on by the nitrifying organisms. Martin, Cameron, and Fowler all expressed this opinion before the Royal Sewage Commission (Martin, 1905). Harding and Frankland (Martin, 1905) are skeptical as to such an advantage and Dibdin (1904) wholly disbelieves it. On the other hand, it is probable, as was shown before the Royal Sewage Commission, that when not accurately regulated, " the anaerobic process may be carried too far, so as to interfere with the subsequent aerobic action " (Dibdin, 1903). Martin and Rideal minimize such interference, while Scott-Moncrieff, Woodhead and Fowler consider it of great importance (Martin, 1905). It appears certain that with strong sewage the putrefactive process may be carried so far that its products will check the aerobic organisms. In experiments at Caterham an effluent was obtained containing 1260 parts per million of dissolved solids, 288 parts of nitrogen as ammonia nitrogen and 54 parts of organic nitrogen, which would not undergo nitrification until diluted (Rideal, 1901). Experience at Andover leads to the same conclusion. Here the sewage is strong and already twenty-four hours old when it reaches the disposal area. Most of it is discharged on sand beds without further treatment. While the beds were successfully handling raw sewage at a rate of 30,000 gallons per acre per day, a small filter gave poor results with septic effluent at a rate of 40,000 and very bad results when the rate was increased to 100,000 (Clark, 1900).

As to the difficulty of treating over-septic sewage in oxidizing filters, Dunbar (1908) states that at Hamburg contact beds could be filled six times a day with fresh sewage without yielding an unsatisfactory effluent, whereas they would only take septic sewage twice a day. It is believed that contrary opinions held by certain authors were due to the fact that septic tank effluent was compared to unsettled and unclarified fresh sewage, which of course contained clogging material.

The experience gained in the study of Ohio plants (Ohio, 1908) suggests that when treating weak American sewage in moderate-sized tanks, little fear need be entertained of rendering the effluent so septic as to be less responsive to treatment in filters.

The opinion that septic action destroys pathogenic bacteria has been occasionally expressed by various observers. This is

true only to a limited degree, and no reliance should be placed upon such an action where sewage is to be purified with a view to protecting a water supply, although any process which removes the suspended matter from sewage, removes to a corresponding degree the bacteria which are attached to the solid particles.

Digestion of Solids in the Septic Tank. The second main criterion for judging the efficiency of a septic tank depends on the liquefaction of the suspended solids. Granting that the tank effects a removal of 60 to 70 per cent of suspended solids under favorable conditions, the fate of the matter retained must next be determined — how much is stored as sludge and how much is reduced to liquid or gaseous form. Evidence before the English Royal Commission indicates widely varying results with different tanks. Watson at Birmingham, after four years of careful study, believes that the digestion of sludge is not over 10 per cent. On the other hand, the following results have been reported at other English towns: a reduction amounting to 26 per cent at Manchester, 20 to 60 per cent at Leeds, 30 per cent at Sheffield, 35 per cent at Accrington, 40 per cent at Huddersfield, 50 per cent at Glasgow, and 80 per cent at Exeter (Martin, 1905). In the London experiments the destruction of total sludge was 41 per cent and of organic sludge 71 per cent (Dibdin, 1903). At Hampton 58 per cent of the organic sludge was destroyed (Baker, 1904). Careful studies made by the Royal Commission, and extending over a two-year period, showed a digestion of 25 per cent at Exeter and 30 per cent at Ilford (R. S. C., 1908). It is rather interesting to note that the original reports from Exeter showed a digestion of 80 per cent against only 25 per cent observed by the Royal Commission.

In the United States the Saratoga tank has been one of the most successful in its operation, perhaps on account of the fact that it treats purely domestic sewage and also because its heaviest burden comes in summer when the septic processes are most vigorous. In the first two years of its operation it received about 1,000,000 pounds of suspended solids and discharged in the effluent, 350,000 pounds. Of the 650,000 pounds remaining in the tank only 200,000 were stored, representing a decomposition of 450,000 pounds or 69 per cent of the solids remaining in the tank. The experimental tanks of the Technology Experiment Station at

Boston received 3790 pounds of suspended solids in the two years, 1905–07; 1120 pounds were retained by the tanks, of which 675 pounds were stored and 471 pounds, or 42 per cent, liquefied. The experimental tank at Worcester removed from the sewage from July, 1900, to Oct., 1902, 1193 lbs. of suspended matter, and in Oct., 1902, contained 729 lbs. in the sludge, giving as the amount liquefied, 39 per cent.

Fuller (1912) cites figures for liquefaction of deposited suspended solids as follows: experimental tanks at Lawrence, 65–88 per cent; experimental tanks at Columbus, 28–67 per cent; experimental tanks at Waterbury, 44–64 per cent; Gloversville plant, 54 per cent; Plainfield plant, 39 per cent.

Metcalf and Eddy (1916) estimate the digestive efficiency of the septic tank at from 10 to 40 per cent and place the average at 30 per cent. They point out, however, that in view of the greater compactness (lower water content) of septic sludge its volume may be expected to be only 20–25 per cent that of sludge obtained from plain sedimentation.

As seems evident from these various analytical results, the amount of sludge digestion in the septic tank varies so widely in different places that it is difficult to make any broad generalizations. The proportion of organic and inorganic material in the sludge must of course markedly affect the results, since only the former can be decomposed. Other variations in the composition of the sewage are also undoubtedly of much importance; it may well be that the poor results reported from Birmingham are due to the presence of harmful manufactural wastes.

Removal of Sludge from Septic Tanks. The frequency with which septic tank sludge must be removed naturally varies with the success of the liquefying process. It is desirable to postpone cleaning as long as possible, on account of the progressive consolidation of the sludge and its gradual decrease in putrescibility. According to the Royal Commission studies, the water content of fresh sludge is 90–95 per cent, while after storage this value may be reduced to 80 or 85 per cent. A decrease of water content from 95 per cent to 80 per cent means only one-fourth the volume of sludge to handle. When, however, the sludge occupies a third of the tank capacity or more, suspended matter is pretty sure to begin to come over in the effluent, and the tank must be cleaned. Even with sludge occupying less than one-

third of the tank capacity, suspended matter will be discharged into the effluent if decomposition and gas ebullition are sufficiently active, as is often the case in the warmer months of the year.

Practical experience in regard to the necessity of cleaning septic tanks has varied within wide limits. At Exeter, Cameron's original tank was operated for eight years without cleaning, but from the present installation, with a fourteen-hour period, sludge is pumped out once a month. Baker reported, from a study of English plants in 1904, that at Barrhead a tank had been in use for six years (twenty-four-hour period) without cleaning and with little deposit. At Acton (sixteen hours) a tank had been operated for fifteen months with no deposit. At Yeovil (twenty-four hours), Burnley (twelve hours), Sutton (five hours) and Accrington (twenty-eight hours) it had been found necessary to remove sludge about once a year. At Oldham the tank was cleaned every two or three months (Baker, 1904). American plants exhibit similar variations. The Mansfield tank was not cleaned for four years after it was installed; but its success in this regard was largely due to the fact that much of the suspended matter entering the tank passed on through it and was not removed at all (Ohio, 1908). At Plainfield, N. J., septic tanks caused much trouble. At Saratoga, on the other hand, sludge was not removed for five years.

Daniels (1914) notes that " some tanks produce a large amount of sludge and little scum, some a very thick scum with little sludge, while others have about equal quantities of both. In one of our tanks after a run of about two years there was no sludge and only about an inch of sand and grit in the bottom. But a very thick scum formed which had to be removed by shoveling off every few months. At another place the writer found less than a foot of sludge in a tank which had been in use for sixteen years."

Metcalf and Eddy (1916) tabulate the results of septic treatment in a number of English and American plants which show periods between cleanings ranging from 9 months at Ashland, Ohio, to 6 years at Lakewood, Ohio. The amount of sludge ranged from 0.10 cubic yard per million gallons of sewage treated at La Grange, Ill., to 15.8 cubic yards at Manchester, England, and from 0.00017 cubic foot per capita per day at

Saratoga, N. Y., to 0.021 at Manchester. The water content of the sludge varied between 80 per cent at Mansfield, Ohio, and 95 per cent at Birmingham, England.

Septic tanks at Washington, Pa. (Pratt, 1908), have been operated for the last 10 years on what may be termed the elastic plan, previously referred to in this chapter. Analyses show that the tanks have been so managed as to get the full effect of sedimentation while limiting septic action so far as possible. The tanks comprise four parallel units each 125 by 4 feet. The method of operation has been to pass the entire flow through one tank at a time leaving three tanks idle. After the accumulation of sludge in one tank becomes objectionably large in quantity and objectionably active in quality, such tank is cut out of service and allowed to stand idle until it is again needed, at which time it is cleaned, the entire contents being drained by gravity onto the sludge area. The results of this operation have been satisfactory, in that the effluent has been sufficiently free from suspended matter to avoid any serious clogging of the sprinkling filters and the sludge almost invariably becomes well digested and inodorous, the quality being similar to that of sludge from two-story tanks.

Septic Tank Scum. One of the most striking differences between individual septic tanks lies in the presence or absence of a surface scum, referred to in the citation from Daniels above. Scum is intimately related to the amount of gas evolved and to the character of the suspended matter in the sewage. The solids composing the scum are carried from bottom to surface by the gas, as it escapes. The material tends at first to sink again, but becomes matted together by means of paper, hair, fat, etc., as well as by vegetable molds, with a result that a tough floating mass is formed which, in some instances, is strong enough to bear the weight of a man. Grass and vegetation sometimes flourish on top of the scum. An analysis of scum from a septic tank at Worcester is cited in Table XLII.

It was formerly believed that scum was essential to septic action. It has been shown, however, that scum is only incidental to such action, and indeed may be distinctly detrimental to it, since the solids in the upper layers of the scum are removed from the sphere of most efficient septic decomposition. The scum is thickest in tanks receiving unscreened sewage, and particularly

TABLE XLII

COMPOSITION OF SEPTIC SCUM AT WORCESTER

	Per cent.
Silica, SiO_2	12.62
Iron sulphide, FeS	8.93
Iron, Fe (not united to sulphur)	1.61
Sulphur, S, not as sulphide	....
Alumina, Al_2O_3	12.62
Lime, CaO	4.23
Carbon, C	41.83
Hydrogen, H	6.51
Nitrogen, N	5.20

if the sewage contains a large proportion of street washings. The grain from horse droppings furnishes an excellent basis for the growth of scum. In a given tank, the thickness of the

FIG. 48. Scum on Septic Tank at Worcester, Mass.

scum varies with the seasons, and with the bacterial activity, although two tanks located in the same climate may have scums differing widely in thickness. At Washington, Pa., the scum at the upper end of a tank receiving unscreened sewage became so thick as to occupy the entire depth. When tanks receive well-screened sewage, or very dilute sewage, little or no scum is

formed. This was the case with the experimental tanks at Columbus.

Starting with a clean tank, the time required to establish septic action varies, as does the intensity of the action, with the temperature, the action, of course, beginning sooner in the summer months. The presence of certain kinds of bacteria is probably necessary to start and to carry on septic action. These bacteria, under ordinary conditions, exist in the sewage and soon establish themselves in the tank. In certain cases, however, as discussed above, these bacteria may be destroyed by acid contained in the sewage, as at Shelby, Ohio (Ohio, 1908). It is believed that the act on may be more quickly brought about by " seeding " a fresh tank with sludge from one already in active operation.

Definite figures, as to the time required for the natural ripening of a tank cannot be given. Frequently one or two weeks will suffice, while on the other hand, in cold weather or with a sewage of unfavorable character, three or four months might be necessary. The Court, in the case of the Cameron Septic Tank Co. *vs.* Saratoga Springs, considered that six weeks was a reasonable average estimate of the time required to establish septic action.

Odors from Septic Tanks. Obnoxious odors may arise from septic tanks or from the effluent passing out of the tank although with small installations very little odor, as a rule, is noticed. As mentioned above, odors, when present, often come from the moist scum rather than from the sewage itself. In England, aerial nuisances from many septic tanks have been the cause of complaint, yet at Birmingham, the largest septic tank installation in the world, odors are practically absent, possibly as a result of the character of the sewage.

Fuller (1912) discusses this question in some detail, particularly in the light of experience at Plainfield, N. J., and Columbus, Ohio. He concludes that at Plainfield " there is ordinarily no objectionable odor noticeable 100 feet away from the disposal works, but that under unusual conditions the odor is noticeable several times that distance, and during the summer of 1910 there were probably six or eight nights when the odor was noticeable a distance of one-quarter mile or more." At Columbus, he states that " the odors ordinarily are not noticeable 1000 feet

away from the plant, but there are a few occasions when the odor is noticeable as far away as one-half mile." It is worth while to note that at both the plants cited the septic tanks discussed have since been reconstructed on the two-story plan.

Septic Tank Patent Litigation. A long and bitter controversy over patent rights has formed an important chapter in the history of the septic tank process in the United States.

It has been pointed out above that the septic principle had long been utilized in England, in France and in the United States for the sedimentation and decomposition of sewage sludge; yet it is certain that the extensive experiments of Donald Cameron had great influence in showing that the principles of bacterial decomposition could in many cases be utilized to better advantage, and on a larger scale, than had hitherto been generally realized. Their practical importance in the history of the process is well stated as follows, in the final report of the Royal Commission (R. S. C., 1908): " The notion that the solid matter of sewage would be digested by passing the sewage through a sealed tank is by no means novel, but it does not appear to have had any extensive practical application until Mr. Cameron, who held the office of City Surveyor of Exeter, proposed the adoption of the ' septic tank treatment ' for that city."

On November 8, 1895, Cameron applied for a patent, and on April 25, 1896, there was granted to him British patent No. 21,142, covering the process of liquefying, by bacterial action, sludge deposited in a flowing stream of sewage; it also covered certain constructional details of design. Although rights under this patent have never been enforced in England, the granting of the British patent led to the issuance of the United States patent, which has been the basis of interesting litigation and discussion in this country.

United States patent No. 634,423 was issued to Cameron, Commin & Martin on October 3, 1899, and soon after became the property of The Cameron Septic Tank Co. of Chicago, Illinois. It contained 22 claims covering both the essential process and the apparatus used; and the license fees charged for the use of the process were so high as to lead to determined efforts to contest the patent rights involved. The first lawsuit of importance concerning its validity was brought against the village of Saratoga Springs, N. Y., in 1904. In Mar., 1907, the United States

Circuit Court of the Northern District of New York gave verdict for the defendants (Saratoga Springs). On appeal, however, this judgment was reversed (Jan. 7, 1908), and the Saratoga tanks were found to infringe claims Nos. 1, 2, 3, 4 and 21, relating to the *process* of septic action. The remaining seventeen claims of the patent were found invalid, as in the decision of the lower court. The opinion of the higher court, in upholding the " Process Claims " read in part as follows:

" We, however, are satisfied that Cameron was the first one to subject a flowing current of sewage to the action of anaerobes and aerobes under conditions which secured their separate and successive action, the action of the segregated anaerobes fitting the effluent for subsequent filtration and aerobic action."

Opposition against the Cameron patents continued and a group of prominent American engineers organized in an " Association for the Defense of Septic Process Suits," and attempted to carry the case to the Supreme Court. This effort failed. Meanwhile, however, a new conflict arose in regard to the life of the patent, the Septic Process League (another organization of engineers opposing the Cameron Co.) maintaining that it expired Nov. 8, 1909, the date of expiration of the British patent on which it was based. This contention was finally upheld by the Supreme Court, Jan. 20, 1913. Nevertheless, litigation has not stopped, because the Cameron Company now claim (1918), that the American patents differ from the original English patent and hence do not come under the above Supreme Court decision and did not expire until October 3, 1916. Furthermore, the Company now claims infringement of the septic process by the users of two-story sedimentation tanks and under certain conditions at least by users of plain sedimentation or settling tanks.

The Present Status of the Septic Tank. There can be little doubt that the introduction of the septic process of tank treatment marked a distinct advance, over plain sedimentation, as regards the digestion and to some extent the liquefaction of sludge, digestion here meaning the changing of sludge to a condition where it could be dried and disposed of without objectionable odors. Without facilities for throwing certain tank units out of service when gas ebullition becomes violent, experience has shown, however, that finely divided sludge may appear in the effluent in great quantities to the detriment of filters; and during

such periods septic tanks are less efficient than plain sedimentation tanks. It is clear that the theory of the septic process as demonstrated in some of the smaller plants, does not work out in general in the larger plants. It is believed that in many early installations the apparent disappearance of sludge could have readily been accounted for if daily determinations had been made of the suspended matter coming out of the tank in the effluent. Nevertheless, septic treatment and the wide discussion which it aroused have had a distinct advantage in promoting progress in the art of sewage treatment.

In 1907, when some of the serious defects of the process were beginning to appear and also when patent litigation was active, a new type of tank, involving separate septic digestion of sludge, came into vogue. This was the Imhoff tank, so-called, which as explained in Chapter VII, was based on a more complete separation of the flowing sewage from the digesting sludge, the need for such separation having been shown by experience with plain septic tanks as discussed above. Reports soon began to paint the future of this substitute for the septic tank in rosy colors. This too was a patented process but the license fee charged for its use was small and the general attitude of the patentee, Karl Imhoff, such as to disarm criticism. Largely through the efforts of Rudolph Hering this process has come rapidly into favor with American engineers. Metcalf and Eddy (1916) state that by the close of 1914 about 75 cities and many institutions had adopted it. The action of Columbus, Ohio, in changing over its great septic tanks to Imhoff tanks after about ten years of use, is significant of the general tendencies of the period between 1910 and 1915.

More recently there has been a certain reaction due to the complaints caused by odors from Imhoff tanks. The Plainfield septic tanks, after being remodeled on the Imhoff plan, continued to give trouble; and at Baltimore Imhoff tanks were abandoned entirely and septic treatment in shallow tanks resorted to instead. In general it seems probable that Imhoff tanks will be likely to prove superior for large communities; but for small installations, at least, the one story septic tank retains a distinct field of usefulness.

CHAPTER VII

PRELIMINARY TREATMENT OF SEWAGE IN TWO-STORY TANKS

Hydrolytic Tanks and the Hampton Doctrine. One of the most interesting modifications of tank treatment for the removal of solids from sewage is the hydrolytic process, developed by Travis at Hampton, England, under the inspiration, as he states, of the experiments at Lawrence, Mass. The work of Travis has not only advanced the practical art of sewage treatment but has contributed in an important degree to a just conception of its theoretical principles. Some of the most interesting features of the Hampton process have nothing to do with septic action, being, in fact, purely physical, rather than chemical or bacteriological. Nevertheless the hydrolytic tank was historically an outgrowth of the septic tank. As a matter of fact, any division line between the processes of sewage purification must be somewhat arbitrary. The septic tank, the hydrolytic tank, the Dibdin plate bed and the contact bed really make up an intergrading series, although the first two are primarily anaerobic and the last two primarily aerobic.

The first step in the development of the Hampton process was the separation of the septic tank into compartments, — two lateral ones, through which the fresh sewage flowed, and a third, between and below the others, into which the sludge settled through special openings and in which it remained until removed. The general arrangement of the original Hampton tanks is shown in Fig. 49. The sewage flowed through compartments shown at *A* and the sludge settled through the openings *B* into the liquefying chamber *C*. The object of providing the separate sludge chamber was to allow sedimentation to go on more freely in the lateral chambers without interference from sludge deposits, from the stirring up due to gas bubbles and from the formation of a thick scum.

Experience with this device led the Hampton workers to focus their attention particularly on the removal of suspended solids. They attempted to carry this further than was possible with the

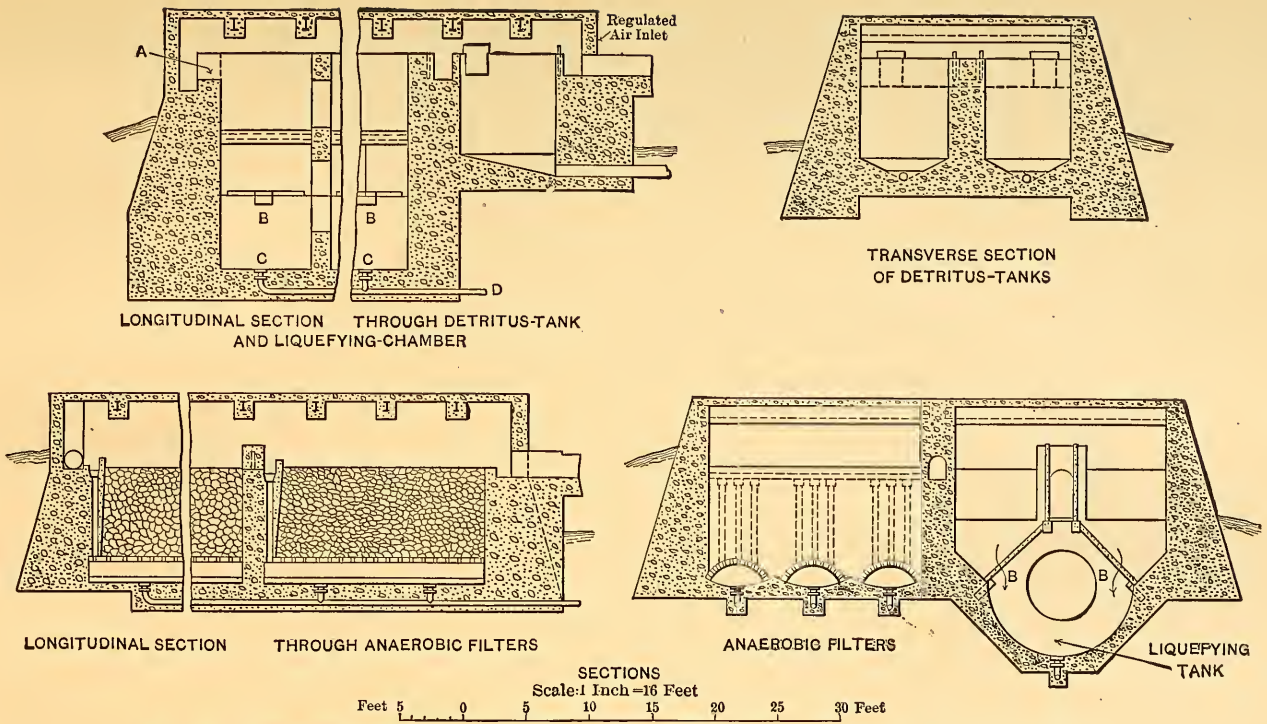

FIG. 49. Sections of the Hampton Hydrolytic Tank and Filters (courtesy of W. Owen Travis).

hydrolytic tank proper (which had a capacity of only 5 hours' flow) by subsequent treatment of the tank effluent in what was at first an anaerobic filter. Four hydrolyzing chambers were provided, filled with broken flints through which the sewage passed by upward filtration. These beds naturally clogged badly, and Dr. W. Owen Travis, who has been mainly responsible for the Hampton experiments, substituted for the flints a series of plates set parallel to each other at a slight angle with the perpendicular. This change was based on the fact that the removal of suspended solids is not necessarily due to real filtration or straining, but to surface contact or adsorption. The evolution of this hydrolyzing chamber from the ordinary anaerobic filter strikingly recalls the development of the Dibdin plate bed from the contact bed. The Hampton tank worked well. The finely divided suspended solids were removed, deposited upon the upper surface of the plates and partially liquefied, so that beneath each plate there was seen a stream of gas bubbles rising, and the excess of the sludge rolled in small masses off the plates and dropped to the bottom of the tank. The arrangement of the plates in a Hampton hydrolyzing chamber is shown more clearly in the figures of the Norwich plant below.

Dr. Travis claimed much more for his hydrolytic tank than the removal of the material originally present as suspended solids in the sewage. He pointed out that a considerable proportion (50 per cent in the case of the Hampton sewage) of the organic matter, ordinarily classed as dissolved because it passes through filter paper, is not in true solution but in the colloidal state. Continued shaking causes these border-line solids to pass into colloidal solution. Standing, and particularly surface contact, on the other hand, leads to de-solution, or the deposition of the colloids in gross suspended form. The hydrolyzing chamber with its plate surfaces thus plays an important part in removing dissolved as well as suspended solids.

Upon these observations and experiments Dr. Travis has developed a far-reaching theory of sewage purification which has come to be known as the Hampton Doctrine and which has been defended by him, by Dr. G. L. Travis, by S. H. Chambers and others in a voluminous series of controversial communications. The Hampton theory of de-solution was first launched by Colonel Jones and Dr. W. O. Travis (Jones and Travis, 1906).

Good statements of the position of the school have been made by G. L. Travis (1908) and W. O. Travis (1908).

Installations on the Hampton Principle. A large plant upon the hydrolytic principle has been built at Norwich, England (Collins, 1908). Its general arrangement is indicated in Figures 50 and 51. It is designed so that the sewage shall pass first through a detritus tank having a capacity of 53,500 gallons and thence to a hydrolytic tank, divided into two lateral sedimentation chambers with a central reduction chamber. The sedimentation chambers communicate with the reduction chamber, below and between them, by narrow ports through which the sludge is to pass. One-fifth of the sewage enters the reduction chamber directly, the amount being regulated by properly adjusted weirs. The capacity of each hydrolytic tank is 260,000 gallons. The effluent from the reduction chamber passes to a hydrolyzing tank with sloping sides and floor having a capacity of 12,000 gallons. The sedimentation chambers of the hydrolytic tank and also the final hydrolyzing tank are equipped with concrete or wooden slabs hung from steel joists, and with their planes parallel to the longitudinal axis of the tanks. Four units are to treat a daily flow of 3,000,000 gallons.

Another plant designed on the Hampton plan has been installed at Luton, England, to treat the sewage of a population of 60,000. Hydrolyzing chambers, reduction chambers and sedimentation chambers are arranged, as described by Tomlinson (1916) in a series of three annular rings about a central sludge well (see Fig. 52). The flow is divided between the various chambers on a somewhat complicated plan, and in certain of the chambers are placed colloidors, in the form of vertical wooden grids of 2 by $1\frac{1}{2}$ inch slats suspended from joists crossing the chambers in a radial direction. The floors are so designed as to slope at an angle of at least 45° with the horizontal in all parts, so that the sludge flows by gravity to the central well and is ultimately removed by an automatic ejector without interrupting the operation of the tanks. The total suspended solids are reduced from 570 to 30 parts per million (94 per cent removal) and the albuminoid nitrogen from 9.5 to 2.8 parts per million (70 per cent removal). The construction cost for the tanks was $9200 per million gallons per day and the operating cost very small (Tomlinson, 1916). There is a small plant of this type in operation at Paqueta Island, Rio de Janeiro (Serber, 1917).

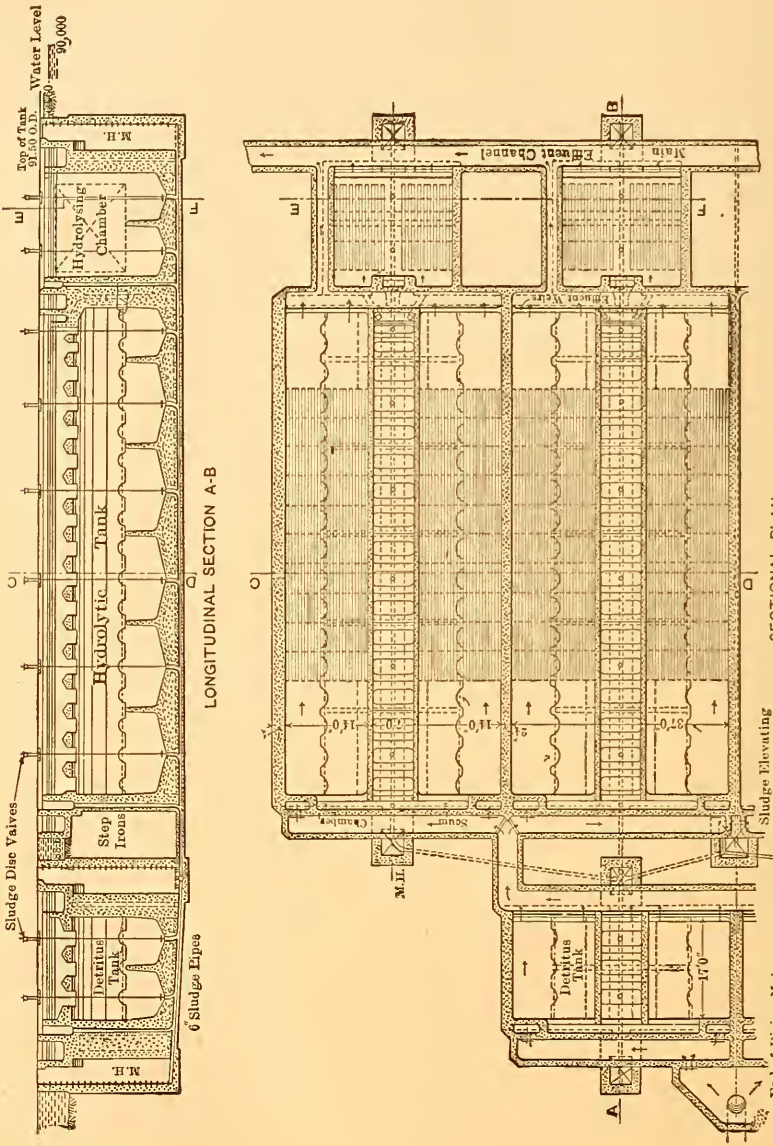

FIG. 50. Plan and Section of Hydrolytic Tank and Hydrolyzing Chamber at Norwich (Collins, 1908).

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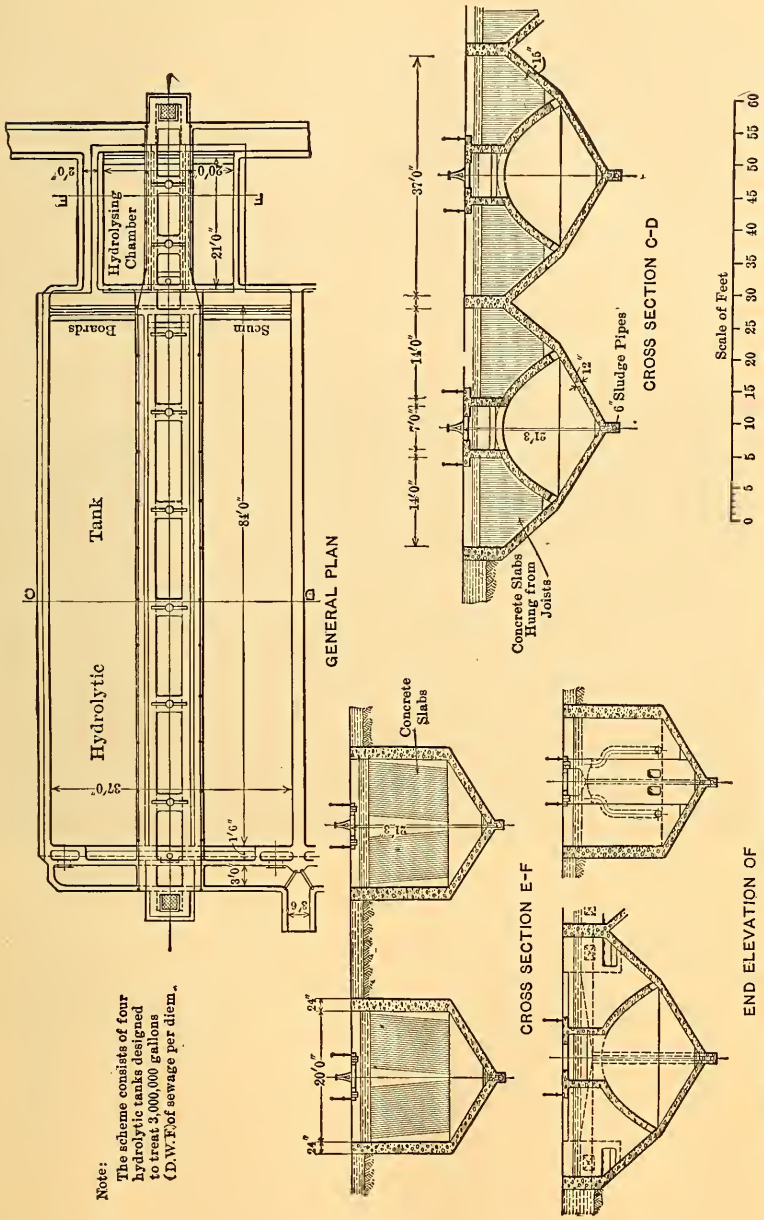

FIG. 51. Details of Hydrolytic Tank and Hydrolyzing Chamber at Norwich (Collins, 1908).

The Imhoff Tank. The most important modification of the septic process is the tank devised by Karl Imhoff for use in the Essen or Emscher district of North Germany, and first placed in operation at Recklinghausen in 1907, — commonly known from its inventor as the Imhoff tank. This tank does not make use of the Hampton principle of the removal of colloidal solids by adsorption; but it closely resembles the Travis tank in the fact that the sludge is collected and digested in a separate lower compartment. This latter aim is in fact attained much more completely than in the Hampton tank. Dr. Travis made special provision for passing one-sixth to one-eighth of the fresh sewage

FIG. 52. Hydrolytic Sewage Treatment Tank, Luton, England (courtesy of Engineering News).

through the liquefying chamber, while in the Imhoff tank every effort is made to separate the sedimentation and the digestion process as completely as possible. In other words, Imhoff adopted and carried further Travis's conception of the separate digestion chamber, but abandoned the idea of the colloidor.

In the tank originally designed by Imhoff, such as is shown in Fig. 53, the sewage flows rather rapidly (in one or two hours) through the upper portion, and is thus kept fresh and free from septic action, while the solids settle through slots into the deep compartment below. Special outlets are provided for the escape of the gases generated in the liquefying chamber so that they do

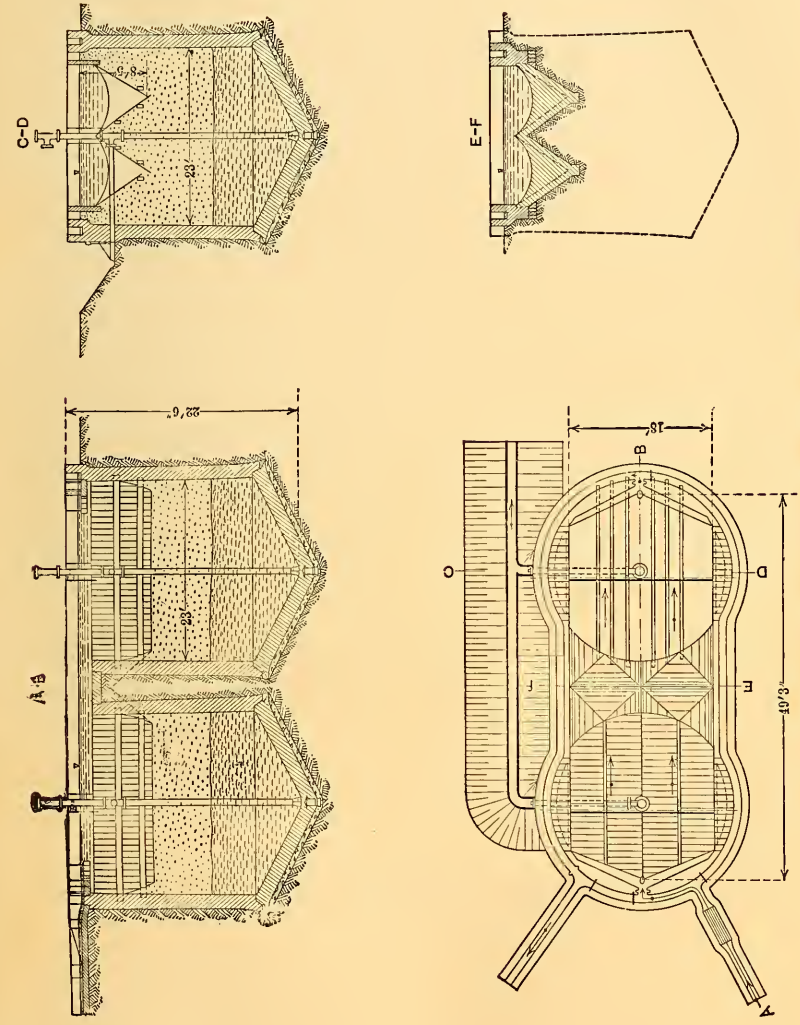

FIG. 53. Plan and Section of the Essen Tank (Imhoff, 1909).

not mix with the sewage stream. Under these conditions the liquefaction of the sludge appears to follow a somewhat different course from that which is characteristic of the ordinary septic tank. The proportion of sludge digested is not appreciably different (Guth and Spillner, 1911), but the residuum is less offensive. The sludge which accumulates is compact and low in water content (75–80 per cent of water) and on account of its content of entrained gases and its granular structure it dries out very readily, and can be disposed of on small areas of land. Most important of all, perhaps, is the freedom from odor of the plant as a whole, which was claimed to be in marked contrast with the offensive conditions which surround many septic tanks. The gas produced in the tank is largely methane and carbon dioxide with little or no hydrogen sulphide. The reasons for this peculiar type of liquefying action are not altogether clear. The fact that septic action is confined to the solids may of course protect soluble sulphates and other putrescible substances from offensive decomposition. It has been suggested that the higher coefficient for the absorption of gases under a greater depth and pressure may prevent the formation of large gas bubbles and consequent discharge into the atmosphere. It is possible that the practices of withdrawing small amounts of sludge every few days and mixing up that which remains by introducing water under pressure, which are in vogue at some of the German plants, may help to promote vigorous bacterial action. Whatever the explanation may be, practical experience in the Emscher valley extending over a period of years indicates that this process may, under proper conditions, be operated so as to yield a reasonably clear and inoffensive effluent, and a compact and inodorous sludge, with no local nuisance even in the immediate vicinity of the plant.

Details of Construction of the Imhoff Tank. The designer of the Imhoff tank in 1916 prepared an admirable review of current practice to that date (Imhoff, 1916). He pointed out as he has done in all his discussions of the subject the importance of maintaining the flowing sewage in the central sedimentation chamber in the freshest possible condition. The thin partition walls of this chamber must therefore be sloped so as to prevent accumulations of sludge. The sludge capacity below can be increased by the use of a longitudinal beam with a slot on each

side instead of a single slot between overlapping partition walls (compare *A* and *B* in Fig. 2 of our Fig. 54); and this is the plan now generally adopted. Distribution, according to Imhoff, should be effected by a 12-inch baffle and a similar baffle should be placed near the outlet end of the tank. The sewage should discharge over a long weir or a number of short ones. Sludge is removed in cleaning without stopping the operation of the tank by such a sludge pipe as is shown in Figs. 1 and 4 of our Fig. 54.

With weaker sewages such as are found in the United States, Imhoff recommends that the sedimentation chamber be made relatively larger and provided with more slots (see Fig. 3 of our Fig. 54). In the United States there is a general tendency to build Imhoff plants on a rectangular plan (see Figs. 55 and 56) though in Germany the circular design is still favored. Downward and upward flow tanks (Fig. 5 of our Fig. 54) have been tried but have not proved very satisfactory.

The sedimentation chamber of the Imhoff tank should be designed for a $1\frac{1}{2}$ to $2\frac{1}{2}$ hours' detention. The capacity of the digestion chamber should be based according to Imhoff (1916) on the data given in Table XLIII below. Gas vents for the escape of gas and the accumulation of scum should occupy about 10 per cent of the superficial area of the tank according to Fuller (1912).

TABLE XLIII

RECOMMENDED SLUDGE CAPACITY FOR IMHOFF DIGESTION CHAMBERS

(Imhoff, 1916)

Cubic feet per capita.

Type of system.	Germany.		U. S.*	
	Separate.	Combined.†	Separate.	Combined.†
Small plants (less than 5000 population)	1.6	2.4	2.4	3.6
Normal city sewage, some trade waste	0.8	1.2	1.2	1.8
Abnormal amount of sludge containing trade waste	1.2	1.8	1.8	2.7

* Northern states with long winters. † Presupposes use of a grit chamber.

Further data in regard to construction are given by Metcalf and Eddy (1916) as follows: The length of tanks should be such as to give an average rate of 108 feet per hour for dry weather flow according to Imhoff, while American practice tends to much slower rates (87 feet at Rochester and 7 feet at Balti-

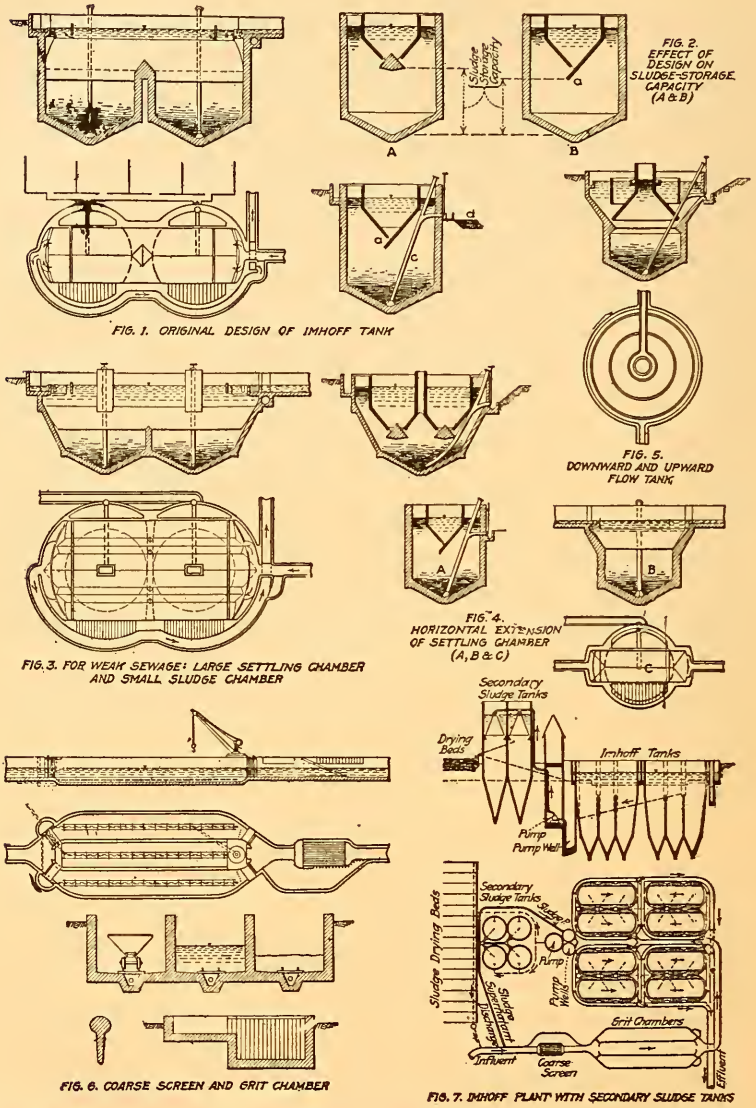
FIG. 2. EFFECT OF DESIGN ON SLUDGE-STORAGE CAPACITY (A & B)
Sludge Storage Capacity
A
B
FIG. 1. ORIGINAL DESIGN OF IMHOFF TANK
FIG. 5. DOWNWARD AND UPWARD FLOW TANK
FIG. 3. FOR WEAK SEWAGE: LARGE SETTLING CHAMBER AND SMALL SLUDGE CHAMBER
FIG. 4. HORIZONTAL EXTENSION OF SETTLING CHAMBER (A, B & C)
Secondary Sludge Tanks
Drying Beds
Imhoff Tanks
Pump
Pump Well
Sludge Drying Beds
Sludge
Supernatant
Influent
Grit Chambers
Coarse Screen
Effluent
FIG. 6. COARSE SCREEN AND GRIT CHAMBER
FIG. 7. IMHOFF PLANT WITH SECONDARY SLUDGE TANKS

FIG. 54. Development of Imhoff Tank Design (courtesy of Engineering News)

more where the tanks were designed on the radial plan). The inclined surfaces of the sedimentation chamber should have a good slope (1.2 on 1 to 1.5 on 1) and a very smooth hard surface. The slot at the bottom of the sedimentation chamber should be from 6 to 12 inches wide and the horizontal overlap to prevent escape of gases at least 8 inches.

The depth of tanks in the Emscher district is generally about 30 feet, and Imhoff believes that depth is a very important factor in producing an easily drainable sludge.

Operation of the Imhoff Tank. The operation of the Imhoff tank, so far as its settling action is concerned, is extremely simple. Imhoff (1916) recommends that where a single sedimentation chamber feeds more than one digestion chamber the direction of sewage flow should be reversed about once in three weeks to equalize accumulation. A by-pass is generally provided for this purpose. The inclined partition walls and slots should be kept free from accumulated solids and it may be occasionally necessary to break up scum or remove floating solids such as pieces of cork or wood.

The frequency with which sludge must be drawn from the digestion chambers varies at different plants, but is stated by Imhoff as generally between two and six weeks. About one month's accumulation should always be retained in the tank to maintain the proper types of bacterial action. In general the optimum position of the sludge level is about three feet below the slot level. The location of the sludge level may be determined by lowering into the sludge chamber in a horizontal position a thin piece of sheet iron one foot square. Gas bubbles or floating sludge in the sedimentation chamber indicate that sludge should be promptly withdrawn.

The success of sludge digestion in the Imhoff process depends on a particular type of bacterial or enzymic action which can only be judged by the results obtained. As a rule a ripening period of about six months must elapse before a typical Imhoff sludge is obtained. "Proper decomposition" according to Imhoff (1916) "is identified by the fact that it occurs entirely without objectionable odors. The effect of such decomposition upon the sludge is that it possesses a not unpleasant odor like that of rubber or warm sealing wax and that it is easily dewatered."

An acid decomposition, which is often noted during the early stages of the life history of an Imhoff tank, is characterized by a foul smelling sludge which is hard to dewater and which often tends to rise in a mass to the surface of the digestion chamber. The addition of lime to the sewage to counteract this acid condition has been suggested by some engineers but Imhoff believes this to be unnecessary and perhaps even undesirable, holding that with proper operation the necessary bacterial flora will be certain ultimately to establish itself. He suggests the hastening of the process by stirring up the scum and sludge deposits with water agitation and, if necessary, by seeding with sludge from a well-ripened tank.

Imhoff (1916) estimates the amount of sludge to be expected from Imhoff treatment at from 0.1 liter per capita per day for a separate system in an ordinary city to 0.3 liter per capita per day for a strong industrial sewage containing trades wastes and street wash. Assuming that a sludge drying bed may be filled to a depth of 12 inches with wet sludge and that this process may be repeated fifteen times a year, the necessary area will vary from 0.2 square foot per capita to 0.6 square foot per capita for the extreme types of sewage cited above. Imhoff's sludge drying areas have been found in actual practice to be too small for American cities in northern latitudes.

The gases given off from the gas vents of the Imhoff tank are as pointed out above frequently almost inodorous; under other conditions this may unfortunately not be the case. At Atlanta the gas was 84.1 per cent methane, 8.6 per cent hydrogen, 4.6 per cent carbon dioxide and 3.1 per cent nitrogen (Hommon, 1914). At Worcester Fales (1913) reports 43.8 per cent methane, 10.2 per cent hydrogen, 27.1 per cent carbon dioxide and 18.1 per cent nitrogen.

American Experiments with Imhoff Tank Treatment. The Imhoff tank has been studied with highly promising results at most of the testing stations where experiments upon sewage treatment have been recently conducted. A small tank of this type at Philadelphia (1911) yielded a well-purified effluent and inoffensive sludge. At Akron (Hommon, 1912) an experimental Imhoff tank effected a removal of 48.4 per cent of the suspended solids of the sewage and digested 25 per cent of the solids deposited. The sludge contained 74 per cent of water against 84

per cent for septic tank sludge, while of the solids present 52 per cent was volatile matter, 0.80 per cent nitrogen and 7.32 per cent fats in the case of the Imhoff sludge, the corresponding figures for the septic sludge being 43.8, 0.96 and 6.50 per cent, respectively.

A particularly comprehensive study of the Imhoff process was made at Worcester (1913). From July, 1911, to December, 1912, an experimental tank treated 40,000 gallons a day with an average flowing-through period of 2.9 hours. The tank removed 57.8 per cent of the total suspended solids, 59.2 per cent of the organic suspended solids, 50.2 per cent of the albuminoid nitrogen, and 44.8 per cent of the oxygen consumed. Scum formed in the scum compartments to a depth of over 5 feet, being at a maximum in warm weather. This scum contained 79.2 per cent water and its dry material was 63.2 per cent organic matter of which 20.9 per cent was ether extract. The sludge produced contained 91.9 per cent water of which 48.5 per cent was organic matter and 6.3 per cent ether extract. The efficiency of sludge digestion may be gauged by the following figures: the sewage influent contained 2984 pounds of suspended solids per million gallons, and the effluent 1427 pounds; of the 1883 pounds deposited in the tank, 1026 pounds were recovered in the sludge and 857 pounds were liquefied. In other words the tank liquefied 45 per cent of the solids it retained.

At Chicago (Chicago, 1914) Imhoff tanks effected about the same purification as other processes of sedimentation (50–70 per cent removal of suspended solids). When operated on the horizontal flow plan a detention period of 1.9 hours removed 58 per cent of the suspended matter, while a detention period of 2.9 hours increased the removal to 69 per cent. The Imhoff tank was distinctly superior to other processes from the standpoint of freedom from aerial nuisance and drainability of the sludge produced. One of the tanks which was most thoroughly studied showed a reduction of 26 per cent in organic nitrogen and 34 per cent in oxygen consumed, while the ammonia nitrogen increased by 33 per cent. Sludge digestion in these experiments was by no means typical, the sludge containing 91 per cent moisture and a high percentage of volatile matter. Nevertheless the proportion of deposited solids digested ranged from 15 to 35 per cent, after the tank had ripened (which took about seven months to accomplish).

At Cleveland (Hoffmann, Pratt and Hommon, 1914) Imhoff tanks effected a removal of 43 to 51 per cent of suspended solids, 21 to 28 per cent of organic nitrogen and 27 to 32 per cent of oxygen consumed, while ammonia nitrogen increased by from 1 to 13 per cent. These results were essentially the same as those obtained by ordinary sedimentation. Short detention periods of thirty minutes effected almost as good results as a period of two hours and fifteen minutes. The Imhoff sludges averaged 85 per cent moisture against 87 per cent for plain sedimentation sludge and 89 per cent for septic and Dortmund sludges; and the volatile matter (32–36 per cent of dry solids in sludge) was less than in the other types of sludge. The results obtained by Imhoff tank treatment of Milwaukee sewage by Copeland (Milwaukee, 1915) are particularly complete and are cited in some detail in Table XLIV below. The detention period used in this tank was about 2.8 hours. Considerable trouble was experienced from accumulations of foul smelling sludge in the gas chambers, sometimes to such an extent as to foam over the sides of the tank. The sludge contained 87 per cent of moisture which dried to a moisture content of 65 per cent in two weeks of dry warm weather.

TABLE XLIV

ANALYSES OF RAW SEWAGE AND IMHOFF EFFLUENT AT MILWAUKEE (1915)

	Parts per million.					
	Total solids.	Suspended solids.	Organic N.	Ammonia N.	Albuminoid N.	Nitrites.
Sewage	1109	259	32.9	15.6	8.9	0.23
Imhoff effluent	957	117	28.8	16.2	7	0.26
Per cent removal	14	54	13	−4	21	.....

	Parts per million.					
	Nitrates.	Oxygen consumed.	Alkalinity.	Chlorine.	Dissolved oxygen.	Bacteria per cubic centimeter at 20°.
Sewage	0.57	125	259	194	2.2	1,461,000
Imhoff effluent	0.51	92	267	170	1.5	1,291,000
Per cent removal	.....	26	.....	.....	.....	12

Removal of settleable solids, 80 per cent.

It is evident that the moisture content of Imhoff sludges varies widely at different American plants. We have noted moisture contents of 74 per cent at Akron; 91 per cent at Chicago; 85 per cent at Cleveland; and 87 per cent at Milwaukee. Metcalf and Eddy (1916) cite records of 92 per cent at Worcester, and 87 per cent and 90 per cent respectively at the two Atlanta plants.

Imhoff Installations in the United States. The acceptance of the Imhoff tank as a practical method of sewage treatment has been almost as rapid in this country as in Germany, in large

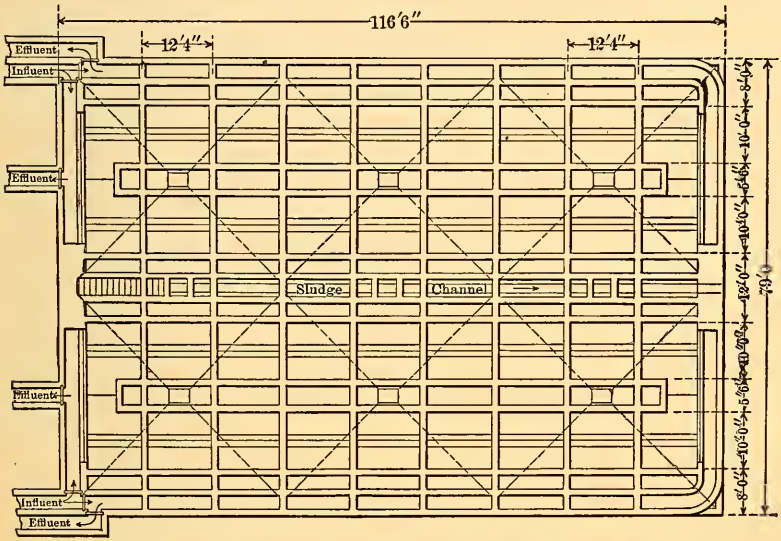

FIG. 55. Plan of Rectangular Imhoff Tank at Rochester, N. Y. (courtesy of C. A. Poole).

measure as a result of the leadership of Rudolph Hering. In 1916 according to Dr. Imhoff 170 tanks of this type had been designed in Germany to serve a total population of 3,000,000, of which 130 were in actual operation, while 70 plants had been designed in the United States to serve a total population of 600,000 with over half of the plants in operation. Among the most interesting of our installations are perhaps those at Rochester, Baltimore, Atlanta, Fitchburg and Columbus.

The Rochester tanks are excellent examples of the rectangular type of construction which has met with most favor in this

country. The tanks (Poole, 1915) are designed in units, each unit to handle the sewage of 40,000 persons after preliminary treatment by 2-inch and half-inch screens and detritus tanks. Each unit (see Fig. 55) consists of two tanks placed side by side, the inside dimensions of a tank being 110 feet by 35 feet with a maximum depth of 40 feet. The sedimentation chamber is made up of two longitudinal V-shaped troughs each 10 feet wide and 12 feet deep from the water surface to the 6-inch slots at the bottom through which the sludge passes down to the digesting chamber (see Fig. 56). The bottoms of these troughs are slabs 5 inches thick, sloped at an angle of 1 horizontal to $1\frac{3}{8}$

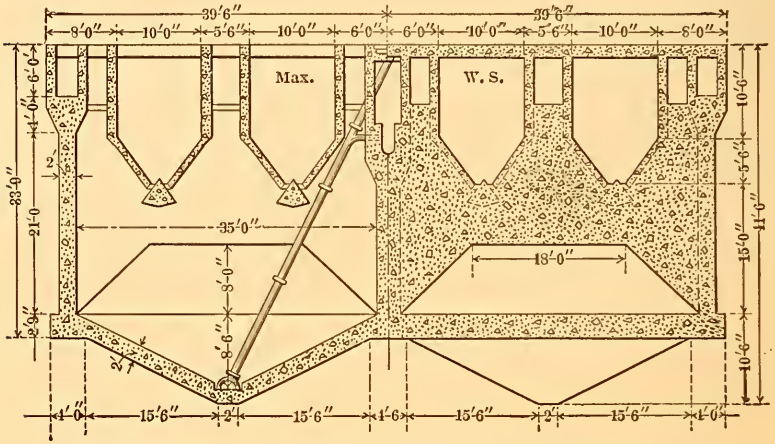

FIG. 56. Section of Rectangular Imhoff Tank at Rochester, N. Y. (courtesy of C. A. Poole).

vertical. Between the troughs is a gas slot three feet wide connecting with the sludge compartment below. Each digesting chamber has three hopper bottoms, each 35 feet square at the top and 8 feet deep, and the distance from the hopper bottoms to the slot in the bottom of the sedimentation chamber is 15 feet, allowing a sludge storage capacity of 2 cubic feet per head of population for a period of 150 days. By-passes (see Fig. 55) permit a reversal of flow. A channel for withdrawing sludge is located between each pair of tanks as shown in Fig. 56. The plant was designed by E. A. Fisher, City Engineer, with the late Emil Kuichling as Consulting Engineer. C. A. Poole, from whose description of the plant (Poole, 1915) these data

are taken, was in charge of construction. The Rochester tanks in their general design resemble the tanks at Fitchburg, Mass., which were among the first designed in the United States though they were not put in operation until 1914.

The Imhoff tanks at Baltimore (Eng. News, 1915) have been built on a different system, known as the radial-flow plan and much used for small installations in Germany. According to Metcalf and Eddy (1916) this type of construction is cheaper and it has the additional advantage of a lower velocity than would obtain in a rectangular unit of the same detention period. The Baltimore unit was a circular one and the influent entered at the center and passed down through a large inverted cone and then upward to adjustable V-shaped weirs on a peripheral channel. Twenty-eight such tanks were provided at Baltimore to care for the sewage of a population of 112,000. Each tank was 40 feet in diameter and 26 feet deep from the sewage surface to the bottom of the digestion chamber. Two hours' detention were provided and a sludge storage capacity of 2 cubic feet per capita (sludge to be stored for six months). After a brief period of operation these tanks were abandoned, on account of foaming in the gas vents and the production of noxious odors. The trouble is attributed locally to the fact that the sewage is old and septic when it reaches the plant.

At Atlanta, Ga., on the other hand, Imhoff tanks have proved very successful (Hommon, 1916). These tanks are of circular type, 24.5 feet deep, and arranged for use in series so as to give a flowing-through period of 3 hours. The tanks at the Peachtree Creek plant removed 49.5 per cent of the suspended solids of the sewage in 1913, 51.6 per cent in 1914 and 59.2 per cent in 1915; while at the Proctor Creek plant where industrial wastes are present which produce a flocculent precipitate the removal values have been better, 76.5 per cent, 84.0 per cent and 85.9 per cent for the three years respectively.

The sludge produced contains 87 per cent moisture and of the dry residue, 39.1 per cent is organic, 1.25 per cent nitrogen and 6.11 per cent ether extract. Some trouble has been experienced from foaming, and large gas vents at a new plant on Intrenchment Creek seem to give worse results than small ones.

An interesting and successful attempt has been made to utilize the gas produced by the Imhoff tank at the Peachtree

Creek plant for lighting and heat in the laboratory and in the home of the chemist in charge. Mr. Hommon estimates that the whole plant is producing 30,000 cubic feet of gas per day or 3750 cubic feet per million gallons of sewage treated. It contains 50–72 per cent methane, 3–31 per cent carbon dioxide and 14–19 per cent hydrogen. At Rock Hill, S. C., a serious aerial nuisance caused by foaming Imhoff tanks has been eliminated according to the Engineering News-Record of Dec. 13, 1917, by collecting and burning the gases produced.

A most interesting development in sewage treatment practice has been the conversion of the septic tanks at Columbus, Ohio (described in the preceding chapter), into Imhoff tanks. It was found at this plant that in order to avoid the carrying over of sludge with the effluent it was necessary during warm weather to remove sludge from the bottom of the old single-story tanks as often as once in four to six weeks. The sludge was highly objectionable and very difficult to dispose of. The tanks were therefore reconstructed on the two-story plan at a cost of $186,000 (Hoover, 1917). This necessitated excavation to a depth of 14.5 feet below the floors of the old tanks. The new tanks are 22 in number, designed for an average flow of 35 million gallons per day. Each tank averages 26 by 130 feet in plan. The average detention period in the sedimentation chamber is 3.5 hours. The digestion chambers have a capacity of 276,000 cubic feet (about 1.1 per capita).

During the autumn of 1916 the first eight of these tanks to be completed were operated at double their normal rate to care for the entire sewage flow while the remaining reconstruction was completed. The result of this heavy burden before the tanks had ripened was very bad foaming in the gas vents, with a diminution of efficiency from a 56 per cent removal of suspended solids in July to 13 per cent in November.

At Alliance, Ohio (Pratt, 1918), an installation of three so-called septic tanks, built in 1910, has recently been modified by converting one of the original tanks into six two-story tanks of the Imhoff type and making minor changes in the outlets of the original tanks so that the sludge deposited in them could be transferred into the digestion chambers of the new tanks. By this plan, which is somewhat novel, the two original tanks were retained for use as plain sedimentation tanks for treating part of

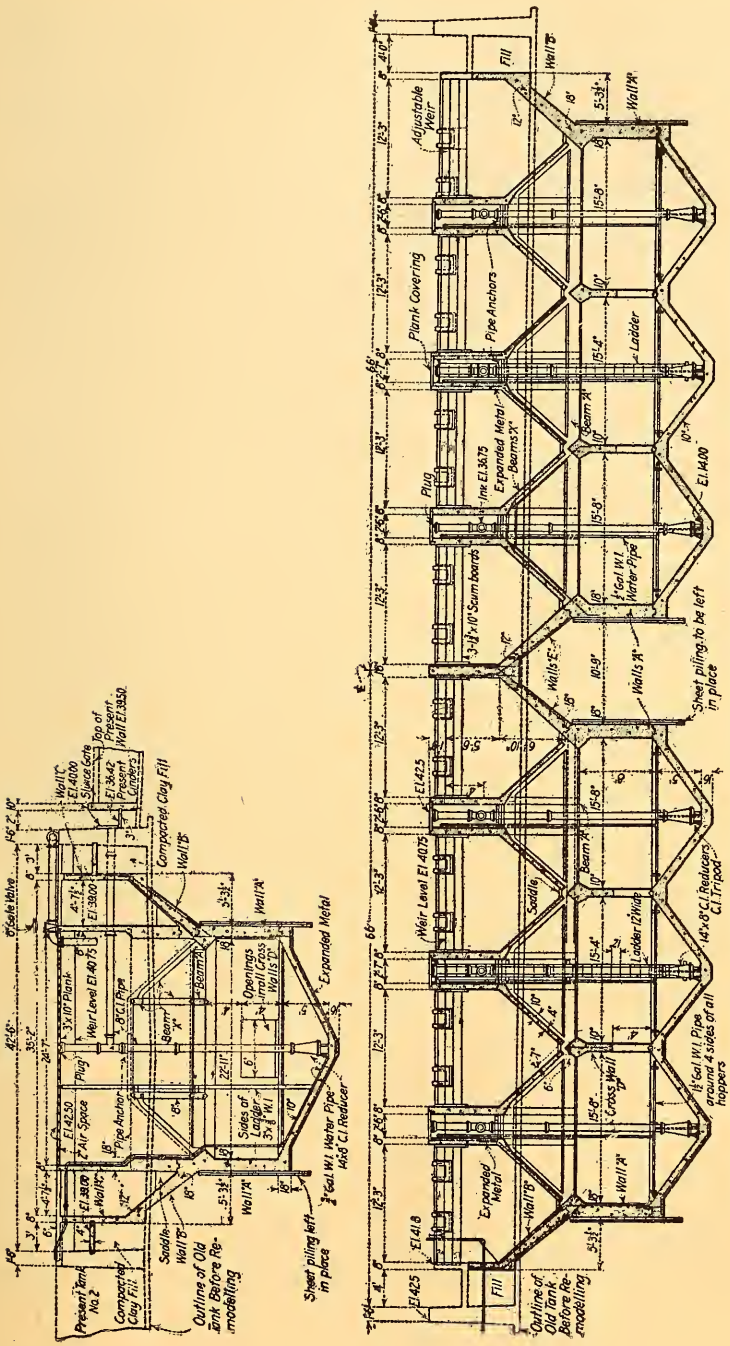

FIG. 57. Transverse and Longitudinal Sections through Two-story Sedimentation Tanks at Alliance, O.

the sewage flow; while the remainder of the sewage was settled in the new two-story tank; and the sludge from all of the sewage is to be digested in these new tanks. Hence, the digestion chambers were of course made larger in proportion to the flow channels than has usually been the practice in America.

Operating Difficulties Experienced with Certain Imhoff Installations. Imhoff tanks were at first supposed to be free from odors, yet the tendency of such tanks to foam at the gas vents and to emit periodic "whiffs" or discharges of foul gases into the atmosphere has caused grave complaints at other places besides Baltimore and Columbus and has led George W. Fuller to recommend the substitution of fine screens for Imhoff tanks in sewage designs at Cleveland, Indianapolis and Bridgeport, Conn. In a recent paper Fuller (1918) has pointed out the special liability to trouble in small plants treating domestic sewage because of the large proportion of gross suspended solids. He points out that "Overflowing from foaming cannot be stopped in some tanks even if the gas vents be extended 5 or 10 feet or more above the flow line. High freeboard, hosing with pressure water, stirring and liming, all tend to help, but collectively may not be a prompt cure. Aggravated cases are not cured by putting a tank out of service for months. In some tanks, sludge removal from the bottom affords a remedy, but this is not available for tanks containing most of the solid matter in a floating condition. The surest remedy in such cases is to remove the great bulk of floating solids from the gas vents until only small quantities remain. Obviously such removal must be done in a way to minimize offense.

"Scum storage for floating solids above the elevation of the slot should be provided for much more liberally than hitherto for small plants, expected to receive during the first year of their operation a substantial proportion of their normal daily quantity of sewage.

"Scum removal may be necessary in some plants even where there is fairly liberal allowance for scum storage. It depends partly on intensity of gasification, partly on the adhesiveness of the scum (gas-retaining capacity) and partly on the success attending efforts to make portions of the sludge remain in the lower part of the digestion chamber. In aggravated cases scum removal is imperative, but it should be carried out with caution."

He emphasizes the fact that sludge storage capacity should be made ample and that sludge bed areas should be double those suggested by Dr. Imhoff — unless covers of the greenhouse type are provided.

The reason why one Imhoff tank works well while another foams badly is not wholly clear; but deficient capacity and lack of skilled operation are usually factors in the problem. The contrast between the two sets of tanks at Rochester, N. Y., is of interest in this connection. The tanks at the large Irondequoit plant treat a stale sewage and have a flowing-through period of less than an hour. While in general, fairly successful, the tanks produce a good deal of odor, and foaming scum in the gas vents must be broken up at intervals by the use of a hose stream. The small Brighton plant on the other hand is working below capacity (running through period over 2 hours) with a fairly fresh sewage; and here the Imhoff tanks produce a high purification with remarkable freedom from odor.

Waldo S. Coulter, Consulting Engineer of New York City, comments upon this problem of foaming as follows in a personal communication to one of the authors.

"From observation I have noted that foaming is usually associated with a 'fresh' sewage, though by no means invariably so; where not chronic, it frequently appears during the early stages of operation, but sometimes manifests itself only after the tank has been in operation for many months, in which latter case such appearances commonly coincide with the beginning of hot weather; that foaming is not confined to acid sludges but may be associated with an alkaline sludge; that frequent withdrawals of sludge tend to retard it; that lime dosage is not of much benefit; that foaming occurs only in conjunction with extremely vigorous gasification; that its occurrence is retarded by ample vents and hastened by restricted vents tending to concentrate the gas bubbles where they escape at the surface of the liquid as, for instance, the chimney type of vent used with the radial-flow Imhoff tank; that a marked viscosity of the liquid is a necessary condition for foaming." Coulter suggests the possibility of dealing with this problem by placing along the sides of the vents downward-acting skimming gates of special design, communicating with glass-covered sludge-drying beds. These gates could be used to run off excess scum at

intervals and at the same time to dilute the liquor in the digestion chamber with fresh sewage passing downward through the slot to replace that removed at the skimming gates. In general it seems that difficulties such as those experienced at Plainfield and Baltimore are the exception rather than the rule and are occasioned possibly by local conditions or details of design. Fitchburg can be pointed to as a prominent plant where no such difficulties have developed. Although most tanks tend to foam at times, this does not necessarily imply the production of a nuisance, and furthermore, it is probable that foaming can be controlled if facilities for drawing off sludge are provided.

The Alvord Sedimentation Tank with Provision for Separate Digestion of Sludge and Scum. John W. Alvord has constructed at Madison, Wis., and also at the Great Lakes Training School in Chicago, a type of tank designed particularly to treat fresh or comparatively fresh sewage, with which there is a tendency toward heavy scum formation. This type of tank has also been proposed for use in connection with the housing developments of the U. S. Department of Labor.

The design which is illustrated in Fig. 58, contemplates first, a separation chamber or series of chambers with ample capacity for the formation of scum or surface floating sludge; second, sufficiently reduced velocities in such chambers to induce suspended matter either to rise and be retained in the body of scum or fall into the hopper bottom; third, inlet and outlet baffles in order to secure diffused inflow and outflow at mid-depth; fourth, a period of detention not longer than is necessary for prompt and reasonably complete separation of the solids from the liquid portions; and fifth, a sludge digestion chamber adjacent to the sedimentation chamber, the level of the liquid in the latter to be maintained lower than the level in the former, so that by means of movable scum weirs the scum from the sedimentation chamber can be pushed over into the digestion tank, while the sludge from the bottom of the digestion chamber can also be drawn off as desired, into the same digestion tank. If additional clarification is necessary a secondary or final sedimentation tank can be installed, the scum and sludge from which is drawn off in the same manner.

It will be seen that one of the chief advantages of the above design is that accumulation of sludge and scum in the sedimen-

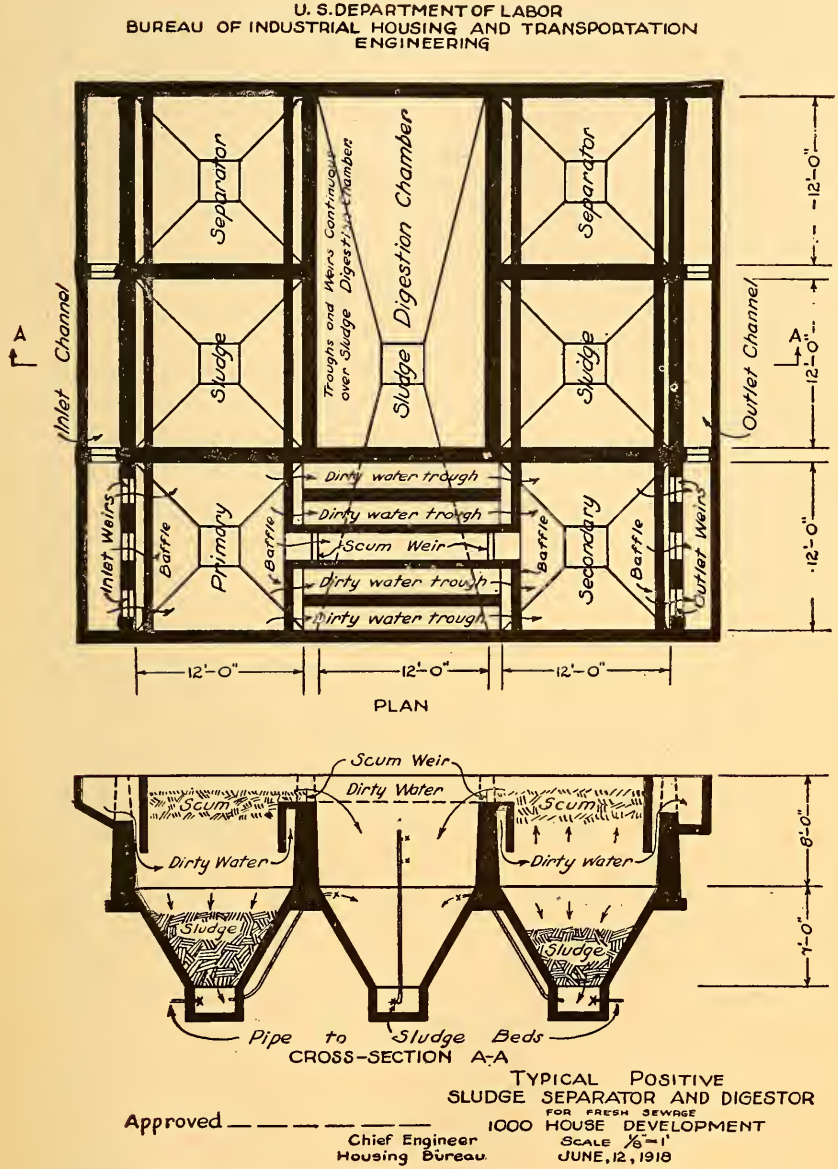

FIG. 58. The Alvord Tank with Separate Sludge Digestion Chamber.

tation tank can be controlled so as not to interfere with clarification of the sewage. This tank, it is believed, embodies the advantages of the Imhoff tank without its disadvantages as regards foaming and fouling of the flow channels.

Comparative Efficiency and Costs of Various Preliminary Treatments. Coarse screening and a small detritus tank, for the removal of the larger and heavier foreign bodies which find their way into any sewerage system, may be considered as essential in all disposal plants, except the very smallest. Beyond this, however, it may be an open question whether any other preliminary treatment is desirable, and, if so, which of the various available methods shall be selected.

Intermittent filter beds can certainly be operated successfully with crude sewage. In the eastern part of the United States, where ample areas of sand of the right quality are available for intermittent filtration, preliminary treatment is rarely held to be necessary. In most cases the solids are discharged directly on the surface, where they are partly oxidized and partly accumulate as dry sludge. In the Middle West, on the other hand, where sand areas are limited and rates of filtration must be high, the use of the septic tank is very general. Barbour, Alvord and Shields are all strong advocates of the practice, and the plants they have installed work very satisfactorily at higher rates than those in use with raw sewage in Massachusetts.

It is not at all certain that it would not be cheaper, even when sand is plenty, to remove a considerable proportion of suspended solids by tank treatment instead of scraping them off the surface of the beds. With rapid filters of the contact or trickling type, thorough preliminary removal of suspended solids is almost universal. Even to this rule, however, it is possible that special exceptions may be made when weak sewages are treated on beds of coarse grain — as suggested by studies made at the Technology Experiment Station in Boston.

If some form of preliminary treatment be decided on, the engineer has his choice between chemical precipitation, plain sedimentation, septic treatment and treatment in two-story tanks. Chemical treatment yields the best effluent, but is most costly. Septic tanks cost more than sedimentation tanks, on account of their greater capacity; but they diminish the sludge to be disposed of. The English Royal Commission made care-

ful cost estimates for the three older processes, assuming a sewage of average composition, ample available head and tanks of simple rectangular design. The principal results of their calculation are tabulated below:

TABLE XLV

COMPARATIVE DATA FOR VARIOUS PRELIMINARY TREATMENTS

(R. S. C., 1908.)

Process.	Tank capacity, gallons.	Suspended matter in effluent parts, per million.	Wet sludge, tons per million gallons.	Cost of preliminary process, per million gallons.	Cubic yards filtering material required for final treatment.	Cost of complete process, per million gallons.
Quiescent sedimentation with chemicals	444,440	10–40	15.5	13.83	4743	20.15
Continuous flow sedimentation with chemicals	833,333	30–60	14.5	12.48	5533	19.74
Quiescent sedimentation	1,041,660	50–80	10.9	7.93	7410	16.38
Continuous flow sedimentation	1,041,660	100–150	10	6–19	9540	16.78
Septic tanks	1,200,000	100–150	5.9	6.91	9540	17.50

All data are in U. S. units.

These estimates are of course very general ones; and the absolute values refer to English conditions only. Nevertheless the table is instructive as a fair illustration of the relative merits of the older processes of tank treatment. It is probably true that septic sludge, allowing for both liquefaction and concentration, will often amount to a little over 50 per cent of the sludge from sedimentation tanks. Chemical sludge on the other hand is nearly 50 per cent more bulky than sedimentation sludge. The cost of the chemical process taken by itself is probably justly estimated as nearly twice that of plain sedimentation. On the other hand chemical treatment produces an effluent freer from suspended solids, and so easier to handle, than the effluent from the other processes. Thus the area required for final treatment is much less after chemical precipitation. The cost for the whole process remains somewhat higher however with chemical treatment and is lowest for plain sedimentation.

The Royal Sewage Commission (1908) recommended chemical precipitation for strong sewages and septic treatment for weaker

ones. The cost of chemicals and the expense and inconvenience of handling large volumes of sludge have however proved serious obstacles to the use of chemical precipitation in recent years. No new plants of this type have recently been installed and many of the older ones are likely to be abandoned in the future. For small plants, particularly those serving institutions or groups of dwellings, the septic tank continues to be the most popular procedure; but for larger installations two-story tanks offer in general the most satisfactory method of preliminary treatment.

Of the two general types of two-story tank the Hampton process continues to be well thought of in England. In this country we have had no experience with this system while the Imhoff process continues in favor. Its chief disadvantage lies in the greater cost involved in the construction of deep tanks. Fuller (1912) states that "the cost of two-story tanks, providing a sedimentation chamber with about a three hours' period of flow and digestion chambers holding about six months' accumulation of sludge, is about double that, generally speaking, of single-story septic tanks having about an 8-hour average period of flow." It is also of course true that the process is a patented one but the royalties so far collected have been reasonable.

The removal of suspended solids effected by the Imhoff tank is about the same as that effected by the septic tank when working at its best without discharge of sludge in the effluent. The organic matter in the effluent is however in better condition and dissolved oxygen is often still present. Above all when in successful operation the sludge produced is more compact, more readily drainable, and much less offensive in nature than the sludge usually drawn from septic or sedimentation tanks. Nevertheless, experience at Washington, Pa., and elsewhere, shows that tanks operated on a combination sedimentation and septic plan may yield sludge quite as satisfactory as that delivered from Imhoff tanks.

During the past two or three years there has been, as noted above, a distinct reaction in some quarters against the Imhoff tank on account of experience which suggests that tanks of this type are by no means as free from odor as had previously been anticipated. The difficulties which have followed from abnormal septic action leading to the foaming up of sludge through the gas

vents and the sudden discharge of large volumes of offensive gases into the air, have been only occasional but when they have occurred have been productive of grave nuisances. Where the removal of suspended solids is not essential and where odors would be particularly disastrous, the substitution of fine screens for any process of sedimentation may sometimes be a wise course. If fine suspended solids must be removed, Imhoff treatment remains the most generally satisfactory method of preliminary tank treatment for city sewage.

CHAPTER VIII

DISPOSAL OF SEWAGE BY BROAD IRRIGATION OR SEWAGE FARMING

The Second Problem in Sewage Treatment — The Oxidation of Its Organic Constituents. It has been pointed out in Chapter II that in the disposal of sewage by dilution there are three main sorts of processes at work, which lead respectively to the separation of the heavier suspended solids and their ultimate disintegration, to the oxidation of the organic materials contained in the sewage and to the elimination of the parasitic bacteria. With a large volume of diluting water and a small amount of sewage these processes may prove entirely adequate without any artificial treatment whatever. With a somewhat higher ratio between sewage and water available for dilution, the oxidative agencies of a stream or lake or harbor may be sufficient to care for the liquid portion of the sewage after screenings or settleable solids have been removed by one or more of the processes which have been described in Chapters III–VII. Where opportunities for dilution are still more restricted it may be necessary to go further and provide special means of treatment which will secure a greater or less degree of oxidation of dissolved as well as suspended organic materials; and the various procedures available for this end remain now to be considered.

The Process of Broad Irrigation. The most natural and obvious method of sewage disposal for communites with no large lakes or rivers near at hand was Broad Irrigation, or the discharge of the sewage upon suitable areas of adjacent land, relying upon the living earth to absorb the liquid and digest the solid constituents of the sewage. The annual disappearance of manure in fertilized land shows how complete this process may be; and it illustrates the fact that the organic matter thus digested is not only rendered harmless, but changed into a form in which it may serve as food material for the higher plants.

The distinguished English sanitary engineer, Baldwin Latham,

believed that he had discovered evidence of sewers and irrigation areas in the ancient city of Jerusalem. It is certain that the Chinese and Japanese have disposed of sewage by this method for thousands of years. The excreta in their crowded countries are either collected in dry closets and spread directly on the fields, or they are discharged into small streams or canals, which are later used for irrigation.

In Europe what was practically sewage irrigation was practiced in a few localities at a very early date. Lausanne, for example, discharged all its sewage into a small brook, which was equivalent to a main collecting sewer. The use of the water of this brook for irrigation was regulated by law in 1400 (Ronna, 1872). In the fifteenth century, too, Milan and other cities in Lombardy constructed canals for carrying their waste waters out to farm lands in the neighborhood. The first irrigation area in Europe designed to take house sewage directly appears, however, to have been at the town of Bunzlau in Prussia. Here, in 1559, a water supply was brought in wooden pipes from a famous spring near by. Sewers were installed, with individual house connections, and the sewage was carried to an irrigation area of about 35 acres in extent. The land was privately owned, and special ordinances were drawn up regulating the hours when each farm should receive its supply of sewage (Du Marès, 1885). For over three hundred years this irrigation area was in use without any sign of deterioration.

Another famous installation is the farm at the Craigentinny Meadows, which receives the sewage of the city of Edinburgh. Irrigation of some sort appears to have been practiced here from time immemorial; but in 1760 an extensive system of sewers was constructed which discharged on open meadows near the sea. The land was originally a waste of sand dunes, but the 250 acres irrigated with sewage have yielded heavy crops of hay and Italian rye grass for a hundred and fifty years.

Sewage Farming in England. The real development of broad irrigation came, however, in England; and it dated from the wave of sanitary reform which swept over that country in the middle of the last century. A. D. Adrian, in testifying before the Royal Commission on Sewage Disposal (R. S. C., 1902), pointed out that the dominant idea of the period from 1842 to 1857 was the necessity for removing excretal matters from the

vicinity of dwellings, without much reference to their ultimate fate. This was the period of sewerage. The report of the Health of Towns Commission in 1844 sounded the first bugle call of the great campaign for public health. It revealed an astonishing accumulation of filth of all sorts in the cities of Great Britain, and the administrative ability of England was at once actively devoted to remedying the conditions revealed. The construction of sewer systems and the elimination of privy vaults and cesspools made rapid progress. Sewerage systems inevitably led to the next step — sewage disposal; and the consideration of this question was the characteristic of the following decades.

In 1857 a new Royal Commission was appointed to inquire into the best means of disposing of sewage (Sewage of Towns Commission). In its first report, in 1858, this commission discussed sewage irrigation with some fullness. The supposed dangers from " the creation of largely extended evaporating surfaces from sewer water," which had retarded the development of irrigation, were minimized. The system in use at Milan, where the liquid refuse of the city was conducted by a canal called the " Vettabbia " to an irrigation area of about 4000 acres, was described as a proper model. In its final report, in 1865, this commission concluded that " the right way to dispose of town sewage is to apply it continuously to land, and it is only by such application that the pollution of rivers can be avoided." According to Mr. Adrian, the dominant note of this period from 1858 to 1870 was the growth of the idea that irrigation, instead of being a menace to health, was the ideal system of sewage disposal.

Even up to 1908 when in its Fifth Report the Royal Commission on Sewage Disposal (appointed in 1898) at last concluded that sewage could be purified by " artificial filters " as well as by irrigation, the Local Government Board of Great Britain held that land treatment was the only proper method of purification. With the desire to dispose of polluting material, there grew up in the early days a parallel interest in sewage farming which was sometimes almost as important as the sanitary one. Liebig and his followers laid great stress upon the danger of an exhaustion of the world's nitrogen supply, and irrigation was hailed as a profitable method of turning organic wastes into valuable crops. The two conceptions are well combined in the definition of ir-

rigation by the British Metropolitan Sewage Commission of 1884 as " the distribution of sewage over a large surface of ordinary agricultural land, having in view a maximum growth of vegetation (consistent with due purification) for the amount of sewage applied."

The construction of extensive sewage farms began at once in England after the report of the Sewage of Towns Commission in 1858, and progress continued at a rapid rate during the next two decades. Ronna, in 1872, described sewage farms at Aberdeen, Rugby, Watford, Carlisle, Penrith, Banbury, Warwick, Worthing, Bedford, Croydon and Norwood, Tunbridge-Wells, Redhill and Reigate, Birmingham, Lodge-Farm, Parsloes, Aldershot and Romford. Du Marès estimated in 1883 that over two hundred irrigation areas of various dimensions were in operation in different parts of England. Many of the sewage farms now in use were laid out in these early days. The farm at Croydon, for example, dates from 1861, and that at the military camp of Aldershot from 1864.

The English enthusiasm for broad irrigation was not slow in spreading to the continent. The first sewage farm operated there on a large scale was at Dantzic. In 1869 a contract was signed with an English engineer, Alexander Aird, by which the sewage of the city and 1300 acres of land were ceded to him for a term of thirty-two years, the entire maintenance of the sewerage system being in his charge. The operation of this plant had a special interest on account of the severe winter weather to which it was subjected; and its success from both sanitary and economic standpoints had great influence in establishing the practice of broad irrigation in Germany. Experiments on sewage farming were begun at Paris by Mille and Durand-Claye in 1865, and important studies of the chemical and bacteriological principles involved were later carried out by Schloesing and Müntz and others. Sewage irrigation at Berlin does not date quite so far back. Operations were first begun at Osdorf in 1876, after a long investigation under the leadership of Rudolf Virchow. An area at Falkenberg was added in 1879, and two areas at Grossbeeren and Malchow in 1882; and the farms are to-day the largest in the world.

Construction of Irrigation Areas. The method of applying the sewage to irrigation areas differs at different farms and in

different portions of the same disposal area. In what is known in England as ridge-and-furrow irrigation the sewage is discharged through numerous ditches from which it seeps out sideways keeping the soil moist but not flooding it. In the commoner practice of ridge irrigation the sewage is flooded over the land from channels and ridges between long gentle slopes, the liquid that finds its way to the bottom of the slope being carried by another set of channels to fields at a lower level. In the earlier farms the intention was that the sewage should pass as a thin film laterally over the surface of the soil and only very gradually find its way downward. Where the soil is porous a third procedure is adopted, flood irrigation, in which the sewage is run to a depth of a foot or more on to a plot of land surrounded by an embankment. This latter form of irrigation is called land filtration in England; but even where this method is adopted the rate of filtration is rarely much over 10,000 gallons per acre per day. Land filtration in England is, therefore, not the same process which we call intermittent filtration in the United States. Here, we limit the latter term to a plant which purifies sewage at fairly high rates (30,000–100,000 gallons per acre per day), generally without cropping; although certain plants like that at Framingham, which have been regularly cultivated, are somewhat intermediate between the American and English types. In England a rate of 30,000 gallons per acre is an exceptional maximum, and the growing of crops is always an important part of the process.

Aside from the laying out of the land the details of the sewage farm are few and simple. As a rule, the sewage is subjected to sedimentation and screening as a preliminary treatment. The Royal Commission in its last report (R. S. C., 1908) points out that " Porous sandy soils, worked as filtration farms, may be able to treat crude unsettled domestic sewage without detriment, but, even in those cases, there is the possibility of nuisance arising from the decomposition of sewage solids on the surface of the soil, and such solids may cause damage to crops."

On the farm itself, the surface must be made fairly level, in order that sewage may not collect in pockets; and when land filtration is used the surface must be carefully evened. The sewage is distributed by open carriers of stone, brick, concrete or half-pipe, the finer ramifications of the system being usually simple

trenches. The importance of proper distribution is well indicated by the history of the sewage farm at Aldershot. From 1880 to 1895 this plant was badly managed and became a nuisance. In the latter year Colonel Jones took charge, and, largely by regrading and by care in laying our distributing channels, brought the farm into such excellent condition that the managers of a military hospital close by are quite satisfied to have it almost beneath their windows.

It is common, as pointed out above, to prepare favorable sandy areas on the farm for treating a portion of the sewage at a some-

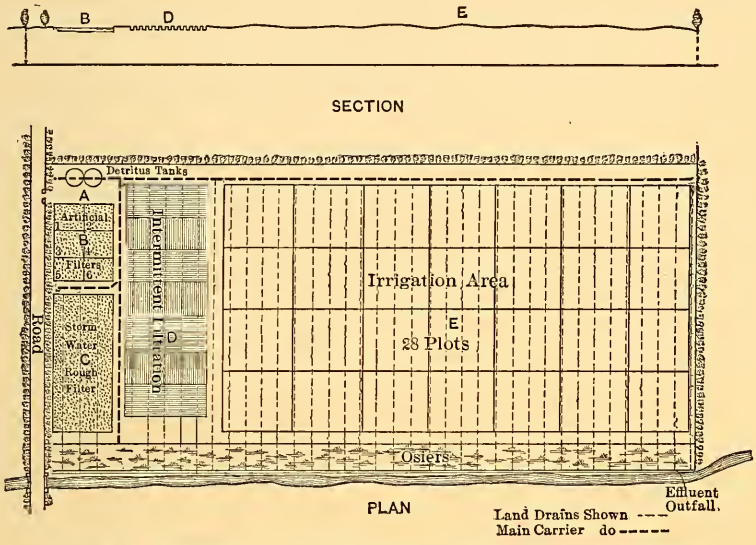

FIG. 59. Diagram of a Sewage Farm (copied by permission from Barwise, 1904).

what higher rate. Provision should also be made for storm water, either by constructing special roughing filters or by allowing an extra area of land. Barwise (1904) gives the diagram reproduced in Fig. 59 as an ideal plan for a sewage farm. The sewage passes from the detritus tanks at *A* to one of the seven intermittent filters or to one of the twenty-eight irrigation plots. Roughing filters at *C* take the first excess of the storm water flow and the osier beds along the bank of the river receive the remainder. If for any reason ordinary concentrated sewage cannot be treated on the beds, it is purified by artificial filters of

crushed stone at *B*, which should have a capacity equal to one day's average flow.

The surfaces of the filtration areas are generally laid out in ridges and furrows, to further facilitate the absorption of the sewage. The distribution from the smaller trenches is controlled by hand, a stop board mounted on a semicircular piece of sheet iron and provided with a handle, being pressed into the soil to dam the sewage stream wherever desired.

It is generally necessary to keep down the ground-water level by digging open ditches at frequent intervals or by laying a system of underdrains; and the treated effluent finally passes into some near-by stream from these ditches or drains. Underdrains are of course to be preferred, except in clay soils, where they may do more harm than good by promoting the formation of cracks and fissures. In England underdrains are laid three or four feet below the surface, and at considerable distances apart, generally from thirty to fifty feet or more between centers. Raikes (1908) gives 660 to 1320 12-inch lengths of pipe as the standard allowance per acre. The gradient of the pipes and their size must naturally be adjusted to local conditions of fall, etc., but 4-inch pipes are the convenient size for the smallest of them. The pipes should be surrounded by fine ashes, gravel or surface soil in order to prevent leakage from above.

The commonest crop on the English sewage farms is Italian rye grass, which is used extensively for fodder as timothy is used in this country. This grass will stand large quantities of water; it is said to yield four good crops a year; and it is so vigorous in growth that it keeps down weeds to a considerable extent. This is an important advantage, for the cutting of weeds is one of the principal operating items in the maintenance of a sewage farm. Rye grass exhausts the land in about three years, however, and must then be temporarily replaced by some other crop. In alternation with rye grass, mangolds (mangel-wurzel), which are beets of a coarse variety, are frequently grown, and are used for cattle food. Osier plantations occupy a portion of the area, and cabbages of various sorts are frequently cultivated. In order to utilize the products of the farm, dairy cattle are often kept and stock-raising interests may even predominate. The luxuriant growth of weeds necessitates constant working over of the land; and at some of the larger plants

steam cultivation is carried out. The direction of a large sewage farm is an administrative task of considerable magnitude.

Operation of Sewage Farms. The operation of sewage farms varies widely, according to the condition of the soil. In some places, as at Altrincham and Nottingham, sewage is discharged upon a given area for twelve hours out of the twenty-four. At Croydon each plot is dosed for a twenty-four hour period every three or four days. In most cases the dosing period is longer still. At Aldershot camp sewage is discharged upon one area continuously for six to ten days, and at Leicester for fourteen days. Whatever may be the dosing period, a corresponding rest must be given, each particular plot being under irrigation for from one-fourth to one-half the time. To obtain an available area of proper soil is the chief problem in sewage farming. A porous soil absorbs a considerable volume of sewage and purifies it as it passes through its minute pores. An impervious soil either becomes quickly clogged or discharges crude sewage through cracks and fissures. A light soil, on a sandy or gravelly subsoil, proves most satisfactory. Peat, chalk and clay, on the other hand, are bad; all three are too impervious and the last two are liable to dangerous cracking. With unsuitable soils rates of filtration must be low. Rideal (1906) estimates that the sewage from 100 persons can be treated on an acre of loamy gravel and that the number may rise to 500 under rarely favorable circumstances, while with stiff clay it falls to 25. The rates commonly in use in England vary from 2000 gallons per acre per day at Leamington and Wrexham to 15,000 gallons at Cheltenham. The evidence collected by the Royal Commission (R. S. C., 1908) pointed to "a maximum rate of 30,000 gallons per acre, or 1000 persons per acre, with the best land, after preliminary treatment, although some witnesses put the rate as high as 60,000 gallons per acre, or 2000 persons per acre, under similar conditions. With unsuitable land, such as clay, not more than 3000 gallons per acre can be efficiently treated, even after settlement of the sewage."

In Germany, the process of sewage irrigation is carried out with greater precautions and, in general, with better results than in England. In the first place, no attempt is made, as in England, to treat sewage on clayey or peaty soils. Irrigation areas are of sand, and almost always carefully underdrained and provided with well-designed distribution systems. Studies at

Berlin have shown that suspended solids, and particularly fatty materials, are highly detrimental to the process. Hence, careful provision is generally made for the removal of suspended solids. The famous sedimentation tanks at Dortmund were designed for use preliminary to irrigation. In some of the German plants, sewage is actually distributed by hand, from hose-pipe nozzles. The rates in general use range from 2000 gallons per acre per day at Brunswick to 7000 at Dantzic, and probably average about 4000 (Bredtschneider and Thumm, 1904).

Fig. 60. View of a "Sewage-sick" Irrigation Area (copied by permission from Dunbar, 1908).

When an irrigation farm is overdosed it becomes "sewage sick," to use an expressive English term (Fig. 60). The surface clogs, pools are formed, putrefaction begins; only a complete rest with thorough plowing restores the health of the area. Under such conditions the temptation to discharge unpurified sewage into the nearest stream is very strong. At times of rain, when sewage flow is highest, the crops are least in need of water and may be seriously damaged by it. The aims of sewage purification are apt, under such conditions, to be sacrificed to those of

agriculture. Thus, at the famous Craigentinny Meadows, where profitable crops are obtained from once barren areas of blown sands, we are told that " the great bulk of the foul water merely runs over the surface of the ground and deposits a portion of its suspended matter " (Barwise, 1904). Many of the English farms, on the other hand, have been operated for thirty years so as to yield a satisfactory effluent without the production of any local nuisance.

Efficiency of Broad Irrigation. The extent of the purification effected by broad irrigation varies very widely, according to the nature of the available soil, the construction of the beds, and the method of operation. If beds of good sand are underdrained and operated as intermittent filtration areas, excellent results may be obtained. On the other hand, sewage which is allowed to run continuously, for long periods, over a clayey soil is certain not to be efficiently purified.

The statistics for eight of the principal English sewage farms compiled in the tables on pages 214–215 from the fourth report of the Royal Sewage Commission (R. S. C., 1904) give a fair idea of general practice:

TABLE XLVI

ENGINEERING DATA FOR EIGHT ENGLISH SEWAGE FARMS

	Aldershot camp.	Altrincham.	Cambridge.	Croydon.	Leicester.	Nottingham.	Rugby.	South Norwood.
Date of construction	1864	1870	1895	1861	1891	1880	1867	1864
Tributary population	20,000	18,000	50,000	100,000	197,000	258,584	6000	21,000
Irrigable area, acres	120.5	35	74	420	1,350	651	35	152
Population per acre	166	514	676	238	146	397	171	138
Dry-weather flow, million gallons	1	0.8	2.25	4	7.25	7	0.3	0.6
D. W. F. per capita	50	44.4	45	40	36.8	27	50	28.5
D. W. F. per acre per day	8,000	23,000	30,000	10,000	5,000	11,000	9000	4,000
Rainfall, inches, per year	22	37	21	23.6	20	25	25.6	24
Storm water treated	Very little. Separate system.	Very little.	Small amount.	All from sewers.	2½ times D. W. F.	Very little.	Nearly all.	All from sewers.
Character of sewage	Domestic.	Mainly domestic.	Mainly domestic.	Mainly domestic.	¼ trade.	⅔ trade.	Mainly domestic.	Domestic.
Preliminary treatment	Screening and settling tanks.	Settling tanks.	Screening and settling tanks.	Screening.	Screening and settling tanks.	Part screened.	Screening and settling tanks.	Screening and settling tanks.
Land treatment	Filtration.	Filtration.	Filtration.	Surface irrigation and filtration.	Surface irrigation and filtration.	Filtration.	Surface irrigation and filtration.	Surface irrigation and filtration.
Soil	Coarse sand.	Peaty.	Sandy loam.	Gravelly loam.	Stiff clayey.	Sandy loam and gravel.	Clay.	Clay.
Subsoil	Very fine sand.	Sand and gravel.	Gravel and sand.	Gravel and sand.	Dense clay.	Gravel and sand.	Stiff clay.	London clay.
Ratio of D. W. F. to flow of river receiving effluent.	1 : 6	1 : 3	1 : 15	1 : 12	1 : 1	1 : 160	1 : 30	3 : 1

Note. — Gallons in this table are English gallons. One English (or Imperial) gallon equals 1.2 U. S. gallon.

TABLE XLVII

ANALYTICAL DATA FOR EIGHT ENGLISH SEWAGE FARMS

Analyses. Parts per million.

	Aldershot camp.	Altrincham.	Cambridge.	Croydon.	Leicester.	Nottingham.	Rugby.	South Norwood.
Total suspended solids								
In sewage	366	720	265	345	341	519	473	219
Total organic nitrogen								
In sewage	50.6	12.7	18.8	20.7	23.4	31.5	32.9	14.9
In effluent	9.4	2.6	2.1	2.6	4.5	0.8	1.7	10.4
Ammonia N								
In sewage	78.7	22.9	38.2	45	81.2	39.8	61.1	35.4
In effluent	18.5	12.3	6.5	14.8	16.5	1.3	16.2	8.7
Albuminoid N								
In sewage	16.2	6.2	9.1	9.1	11.9	14.5	17.3	6.7
In effluent	2.6	1.3	0.9	1.4	2.0	0.3	1.8	1
Nitrogen as nitrites and nitrates								
In effluent	32.1	.....	12.6	3.7	5.1	20.6	5.2	3.9
Oxygen consumed 4 hrs. at 80°								
In sewage	207.9	52.2	100.8	124.8	223.5	232.1	184.4	77.1
In effluent	27.2	17.5	8.1	14.1	25	1.9	14.1	14.4

The low rates on clayey soil at Leicester, Rugby and South Norwood will be noticed, as well as the fact that careful screening and settling has in most cases been found a necessary preliminary. The analyses, which from their source may be considered representative, indicate that English irrigation effluents are by no means of exceptional quality. The Nottingham results are excellent, and those obtained at Cambridge, fair. The Aldershot plant appears to be doing good work, in view of the very strong sewage with which it deals. At Altrincham, on the other hand, a weak sewage is not well purified, and the effluents obtained at

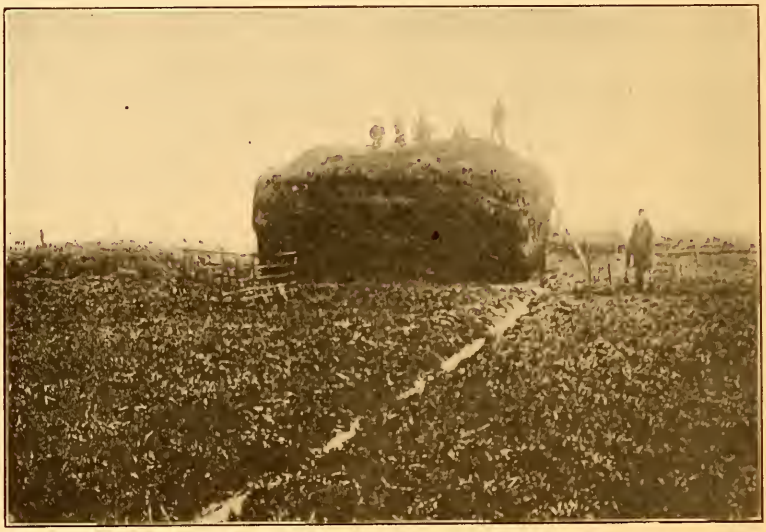

Fig. 61. Getting in the Hay Crop on an English Sewage Farm.

Croydon, Leicester, Rugby and South Norwood can scarcely be considered satisfactory. The most important factor appears to be the nature of the soil, and its necessary corollary, the method of operation. At Aldershot, Cambridge and Nottingham, sandy areas are operated by downward filtration alone. Leicester, Rugby and South Norwood stand at the other extreme; their soils are mainly clay and the sewage is treated largely by surface irrigation.

In regard to organic stability, as determined by incubator tests, the experts of the Royal Commission reported that samples of the effluent at Nottingham never putrefied, while the Cam-

bridge effluents also stood very high, and of the samples from Leicester and Aldershot 90 per cent gave no secondary putrefaction. Norwood, Croydon and Rugby, on the other hand, gave putrescible effluents about one-fourth of the time. On the whole, it seems fair to conclude from a general survey of English conditions that when a sufficient area of porous soil, with a low-water table, is available a well-managed irrigation area may yield effluents of considerable organic purity.

With regard to bacterial removal similar differences appear.

TABLE XLVIII

BACTERIAL EFFICIENCY OF ENGLISH SEWAGE FARMS
(R. S. C., 1908.)

Bacteria per c.c. in effluent.	Aldershot.	Altrincham.	Cambridge.	Croydon.	Leicester.	Rugby.	So. Norwood.
Gelatin, 20°.....	183,266	263,400	711,476	1,413,200	532,777	637,133	778,322
Per cent removal compared with sewage.....	99	99	94	95	95	97	98
Agar, 37°.....	37,308	7,275	78,327	112,000	70,500	81,526	35,157
Per cent removal compared with sewage.....	99	99	94	97	95	97	99
B. coli, approximate	1,000	100	1,000	1,000	1,000	1,000	100

The table above indicates the general results obtained in the studies of the Royal Commission. Average figures for Nottingham are not given, although it is stated that the total number of bacteria in the effluent was frequently less than 1000 per c.c. and that the removal of *B. coli* was usually very satisfactory. In general the table shows that the number of bacteria in the effluents is very high. In some cases even this high value represents a good per cent purification, as measured against the very strong sewages applied. In other cases, however, the per cent purification is low; for 94 or 95 per cent is a poor grade of purification when dealing with such large absolute numbers of bacteria as are found in sewage.

Sanitary Aspects of Sewage Farming. The success of any purification process, from a sanitary standpoint, must be judged in two ways. The final effluent produced should be of satisfactory quality, and conditions about the plant should be such that

no serious local nuisance is created. In regard to the first point, it seems clear that the efficiency of broad irrigation depends entirely upon the nature of the soil and the method of operation. On the whole, it may be said that a sewage farm, under the best conditions, may yield a better effluent than can be obtained from contact beds or trickling filters. Where the soil is heavy, however, the results of irrigation are very much inferior to those which can be attained by the so-called " artificial " processes. Furthermore, in sewage farming there is always a tendency to the by-passing of surplus sewage at times of rain, and in many instances this militates seriously against the general efficiency of the process.

In regard to the second point, there is no reason to anticipate any serious local nuisance from a properly operated irrigation area. Prior to the report of the Sewage of Towns Commission in 1858, there was a general fear that sewage spread out on the surface of the land would breed the miasms of disease. There is often an appreciable odor on the sewage farm itself, and disease germs might conceivably be spread by insects and in other ways from any sewage disposal area. On the Berlin farms, however, there is a resident population of four thousand persons, yet the careful observation of the German authorities has failed to show any detrimental influence upon health. It is of course obvious that well waters in the neighborhood of irrigation areas must be subject to strict supervision. Recent studies in England have shown that specific bacteria may pass for a distance of two miles in less than three days through chalky soil of a porous nature.

Finally, there is one other sanitary problem to be considered in connection with sewage farming — the possibility that infection may be spread by the crops grown on the fields. Where vegetables are eaten raw, after irrigation with sewage there may, no doubt, be serious danger. An epidemic of 63 cases of typhoid fever at Northampton (Mass.) Insane Hospital in 1899 was pretty clearly traced to celery manured with sewage sludge (Mass., 1900). The English experts, McGowan, Houston and Kershaw (R. S. C., 1904) would even go so far as to limit sewage farms to the raising of food for cattle: " We are, on the whole, not in favor of sewage farms being utilized for the raising of crops for human consumption."

This seems an extreme position. The cultivation of fruits and vegetables which grow near the ground and which are to be eaten raw should certainly be prohibited. With this restriction, however, there seems no reason why irrigation areas should not be cropped for the market. The long experience of Berlin and Paris indicates that there need be no serious danger of the spread of disease from irrigated crops, under such conditions.

Economic Results of Sewage Farming. The economic advantages of sewage farming have long been a debated question. The utilization of waste products is always an attractive idea; and sewage fields covered with a rich mantle of luxuriant vegetation make a strong appeal to the imagination. It has been said, however, that it is poor economy to save something by a process which costs more than the value of what is to be saved. English chemists estimate the manurial value of sewage at from 1 to 4 cents a ton (Rafter and Baker, 1894). This value can no doubt in part be recovered, since the crops grown on sewage fields are often astonishingly heavy. Whether it really pays to recover it, however, is another question; and the answer entirely depends upon varying local conditions. The fertilizing elements of the sewage are in a highly diluted condition. Much of the organic matter present is in a form not readily available for the use of plants; and the fats, soaps and other clogging materials present may so injure the soil as to more than neutralize any fertilizing value. In many English towns the operation of farms has proved unprofitable, and there is a general tendency toward their abandonment. Lieut.-Col. A. S. Jones of the Aldershot farm and others have been, however, ardent advocates of the process; and it appears that this farm, besides paying all running expenses, yields in some years as much as $12 an acre toward rent (Baker, 1904).

The British Royal Commission in its final report (R. S. C., 1908) gives some careful estimates, which indicate pretty clearly the relative cost of land disposal under different conditions. The actual figures are of course not comparable with American figures of to-day but the increase in expense with poorer soils and the extent to which the returns from crops may be made to pay the cost of disposal are fairly indicated in the table below. The estimates are based on a flat price of $484 per acre for land and a laborer's wage of $5.04 per week. They include provision for

preliminary sedimentation and sludge disposal, as well as for construction and operation of the beds themselves.

Comparative costs for percolating beds range from $19.74 to $24.28; and for contact beds, from $27.28 to $35.28.

TABLE XLIX

ESTIMATED COSTS AND RECEIPTS. SEWAGE FARMING IN ENGLAND

Per million gallons.

Nature of soil.	Process.	Gross cost.	Receipts.	Net cost.
Sandy loam overlying gravel and sand	Filtration with cropping	$16.45	$1.65	$14.80
	Filtration, little cropping	12.02	0.78	11.24
	Surface irrigation with cropping	20.27	2.88	17.39
Heavy soil overlying clay	Surface irrigation with cropping	29.21	3.96	25.25
Stiff clayey soil over dense clay.	Surface irrigation with cropping	41.47	6.62	34.85

There are two practical questions upon which these estimates throw light. The first problem which confronts any community contemplating sewage disposal is the alternative between land treatment and the so-called artificial processes (contact beds and percolating beds). The figures show that land treatment is cheaper where there is suitable sandy soil available, but that percolating beds are more economical than treatment on unsuitable land. Granting that land treatment is indicated, the next question is whether the area should be used for sewage disposal alone or whether the treatment of sewage should be combined with cropping. McGowan, Houston and Kershaw, in their valuable report to the Royal Sewage Commission, conclude:

"Although we are of opinion that sewage farms in general can never be expected to show a profit if interest on capital expenditure is included, the fact that in favorable seasons some of them more than cover the working expenses is a point in favor of cropping in connection with the land treatment of sewage" (R. S. C., 1904). On the other hand, according to the table of estimates cited above from the final report of the Royal Commission, the net cost of filtration with cropping is greater than that of filtration on similar land with little cropping.

The Sewage Farms of Paris and Berlin. The most notable examples of broad irrigation on a large scale are the sewage farms of Paris and Berlin. Irrigation was first suggested at Paris by Mille as far back as 1865. It met at first with much opposition on theoretical grounds; but after valuable preliminary investigations, to which reference has been made above, an experimental study was carried out in 1868, at Gennevilliers, in comparison with chemical precipitation. The land treatment proved entirely successful. When it came to the point of laying out sewage farms on a large scale, however, there developed a political opposition on the part of the communities where the sewage was to be distributed, which proved a more serious obstacle than the earlier scientific doubts of the efficacy of the process. This obstacle was not overcome until 1889, when a law finally passed the French parliament permitting the general development of the irrigation scheme. A new fight was made against loans for actual construction; and the farms were not extensively laid out until 1895 (Surveyor, 1900).

The Paris plant consists of four large areas, the two oldest at Gennevilliers and Achères and two of more recent origin at Pierrelaye and Carrières-Triel. The total irrigated area amounted in 1905 to 13,597 acres, of which 4359 acres were owned by the city (Calmette, 1907). With 185,000,000 gallons a day, the net rate was about 12,000 gallons per acre. The land is for the most part of excellent quality, largely the ancient alluvium of the Seine. The sewage is carefully sedimented and screened at the Clichy pumping station. At the farms it is distributed from concrete outlet chambers by open ditches. At Gennevilliers, the main channels of masonry or concrete are 39 to 49 inches in diameter, and their smaller branches are 12 to 23 inches in diameter. On this farm alone there are 34 miles of distributing channels besides the final ramifications of the earth furrows. Concrete underdrains are laid 13 feet below the surface (Surveyor, 1900). About a third of the land, as noted above, is owned by the city, but most, even of this, is privately operated under contract. At Gennevilliers the land is privately owned and the use of sewage is entirely voluntary, so that the demand for it is a fair indication of the success of the process. The city maintains, however, a model experimental garden in which various fruits and flowers are cultivated. On privately owned and operated land

the responsibility of the city ceases at the outlet chambers. Large portions of the farms are used for pasturage; and among the crops grown are peas, artichokes, tomatoes, onions, potatoes, asparagus, sugar beets and cereals. The cultivation of strawberries, salad crops and other foods which are freely exposed to the sewage and then eaten raw, is prohibited. It is an interesting commentary on the economic aspect of sewage farming that the city has recently had serious difficulty in securing extension of its farms on account of the opposition of farmers who objected to the unfair advantage given to the favored few who would receive sewage for irrigation. At the Gennevilliers farm (of 2000 acres), the crop is worth over $400,000 a year. It is maintained by competent authorities (Calmette, 1907) that the farms as a whole could be operated far more efficiently than at present if the agricultural aspects of the case were more intelligently kept in view. Asparagus, for example, is a crop which is not at all adapted for intensive sewage treatment, and timber land is also wasteful on an irrigation area.

There are no published data by which the financial success of the Paris farms can properly be judged. The total cost of the farms up to 1900 was $7,220,000; and the annual operating expense nearly a million dollars a year (Surveyor, 1900). A part of this sum is made good by the sale of crops and the rent of irrigated land. At Gennevilliers the farmers use the sewage freely, and there is no return to the city at all. At the other farms the rent of the land is doubled or trebled by irrigation. The profits altogether are small, however, and the process as a whole a costly one (Calmette, 1907).

Of the whole volume of sewage applied to the Paris farms, fully one-half reappears in the drains; and regular analyses, made at the municipal observatory of Montsouris, show that the purification attained is excellent. The figures in Table L are fairly typical:

TABLE L

ANALYSES OF SEWAGE AND EFFLUENTS AT PARIS

Parts per million. (Calmette, 1907.)

	Organic matter.	Nitrogen as —	
		Nitrates.	Ammonia.
Sewage. Clichy	43.3	0.3	22.0
Effluent. Gennevilliers	1.0	31.1	...
Achères	1.7	17.9	0.5
Pierrelaye	0.8	14.2	...
Carrières-Triel	1.2	26.2	...

Bacterial results are less satisfactory. Frequently the numbers are down to a few hundred per cubic centimeter; but at other times they may rise to hundreds of thousands. The most serious shortcoming of the Paris farms is, however, their inadequate capacity, which makes it impossible for them to handle the heavy sewage flows at the time of the spring rains. In such periods 35 per cent of the total sewage may be discharged, unpurified, into the Seine.

The Berlin sewage farms offer examples of broad irrigation under better conditions. The four farms at Osdorf, Falkenberg, Grossbeeren and Malchow were laid out between 1876 and 1882. Three more have since been added, at Sputendorf, Blankenfelde and Buch. There were altogether in 1910 (N.Y., 1914), about 43,000 acres on these farms of which about half were actually irrigated. Of 22,001 acres receiving no sewage at that time 10,647 acres were farmed by the city, 2486 acres were leased to farmers and 8868 acres were classed as unproductive land. Of 21,008 acres receiving sewage 16,657 acres were farmed by the city, 3956 acres were leased to farmers and only 395 acres were unproductive. The contributing population at this time was 2,064,000 and the average amount of sewage treated was 77,000,000 gallons, giving a daily rate of treatment of about 3700 gallons per acre of prepared land. The soil is sandy and of excellent quality. A quarter of the area operated by the authorities is devoted to pasturage, and about a third to the cultivation of cereals, of which winter rye and oats are the most important. Potatoes and beets are grown in considerable amounts and a wide variety of other crops in smaller proportions. Access

to the markets is convenient for most of the farms, and dairies and distilleries are maintained for immediate utilization of the crops. Even fish ponds are made to yield a part of the revenue, and the drains on some of the farms have been successfully stocked with brook trout.

The Berlin irrigation system has been notably successful from the sanitary standpoint. Suspended solids and fats are carefully removed from the sewage before it is applied to the beds. The farms themselves are well kept and exhibited as show places for visitors; at the Blankenfelde and Buch areas playgrounds are maintained for the school children of the city. The chemical purification attained in the effluents from the farms is indicated in the table below. Results are continually being improved by the construction of sedimentation basins for the effluent and by the laying out of special areas for secondary filtration.

TABLE LI

COMPOSITION OF SEWAGE AND EFFLUENTS AT BERLIN

Parts per million. (Dunbar, 1908.)

	Total residue.	Loss on ignition.	Oxidizability, potassium permanganate.	Chlorine.	Ammonia and albuminoid nitrogen.	Nitric acid and nitrous acid.
Sewage	978.4	285.2	333.7	283.8	99.5	....
Effluent	987	124	33.6	232.7	2.3	146.6
Per cent purification	....	56	90	10	98	....

The bacteria in the effluents averaged 34,000 in forty-six samples examined in 1906. Individual counts ranged from 15 to 417,300 (Berlin, 1907).

The cost of the Berlin farms to March 31, 1910, was $17,470,000, somewhat more than half being the purchase price of the land. The expenses for this year amounted to $1,300,385 for maintenance and $741,818 for interest charges. The receipts were $1,240,773 and there was an estimated increase of $122,593 in value of live stock and other property. The farms were operated in part by convict labor; but the living expenses of the convicts were paid and every gang of ten was in charge of a high-priced guard, so that the net expense for labor was not materially reduced.

The Sewage Farms of Moscow. Another European capital which disposes of its sewage by irrigation is the city of Moscow. This is one of the few Russian cities which have developed a comprehensive system of sewerage and sewage disposal. Even in Moscow only the central district of the city is at present connected with the sewers, and in 1915, 572,442 cartloads of night soil (averaging 28.5 poods or 1,026 pounds per load) were removed from the outlying districts and dumped, under highly offensive conditions, in areas of low land. The city has been divided into two zones for sewerage purposes and the sewerage of the inner zone was begun in 1892 and completed in 1898. Work on the second zone was begun in 1910. The system is built on the separate plan and its cost to 1913 was 14,523,915 roubles. On Jan. 1, 1916 there were 7465 properties connected with the sewers, 438 versts (292 miles) of sewers, and a mean sewage flow of 6,768,000 vedros (18,275,000 gallons) per day. This amounts to between 7 and 10 gallons per capita on the basis of the actual contributing population (probably not over a third of the total population of the city).

The sewage of Moscow is at present disposed of by irrigation at two different areas, one at Lubline, which treats about two-thirds of the total amount (about fifteen million gallons in the summer of 1917) and another at Luberzy, which handles the remainder and provides space for expansion in the future as the outer zone of the city is gradually connected. The Lubline farms are about 8 miles from the city, on the left bank of the Moscow River. They include 1161 dessiatins (3135 acres) of land, of which about a tenth is under cultivation, cabbages, rye grass and willows being among the principal crops. The main part of the area is not cropped at all. Of the total area, about half is clay, a quarter peaty soil, and a quarter sand, and operating results differ materially, as would be expected, on the different soils. The clayey and peaty areas treat about 3000 vedros per dessiatin; the best of the sand areas about 25,000. (Since a vedro equals 2.7 gallons and a dessiatin 2.7 acres, vedros per dessiatin and gallons per acre are interchangeable terms.) The clay lands are low, and are flooded by the river in spring. Doses of sewage of about 8 inches in depth are applied to a given area at intervals of from 4 to 10 days, depending on the character of the soil. 2484 acres are low enough to be fed by

gravity, while the sewage is pumped to the remaining sections. The sewage pumped to the sand beds (essentially intermittent filters) which are on a higher level than any of the others, is submitted to bar screening ($2\frac{1}{2}$ cm. mesh) and brief sedimentation (10 minutes). The rest of the sewage receives no preliminary treatment.

The irrigation fields are divided into units of from 3 to 22 acres, and the farms are provided with some 80,000 linear feet of brick and concrete distributing channels, 2700 feet of wooden

Fig. 62. Distributing Ditch, Moscow Sewage Farm.

channels, 590,000 feet of earth distributing channels, 117,000 feet of drainage channels, 442,000 feet of embankments, 277,000 feet of roadways, and 2,088,000 feet of drains, of which the greater part are 3 inch tile. One of the higher level beds with its irrigation ditch is seen in Fig. 62.

The construction costs of the irrigation fields to 1911 are given as 2,233,365 roubles and their operating costs as 387,000 roubles a year.

The Luberzy area which was purchased in 1910–11 includes 1624 dessiatins (4385 acres) and will care for 15,750,000 vedros

(42,500,000 gallons) a day according to the estimates of the authorities.

Sewage Farming in the United States. Sewage farming has been attempted at various small institutional plants in the eastern and central states, as a rule only to be abandoned after a brief trial. The growing of crops on intermittent filters has been practiced in some instances but the clogging of the soil as a result of the accumulation of humus material has proved detrimental. It is only in the arid regions of the West, where the water value of sewage comes into play, as well as its manurial value, that broad irrigation has undergone any important development on this side of the ocean.

In regions with a more or less desert climate it is obvious that all the conditions for sewage farming are particularly favorable. Sewage farms were laid out in Wyoming, Colorado and Nebraska before most of the eastern cities realized that there was such a thing as a sewage disposal problem. In some of the western states the total annual rainfall is only 5–15 inches and there is no rain at all for eight months of the year. Under such conditions the case for irrigation is almost irresistible.

The first of the western irrigation plants was laid out at Cheyenne, Wyo., in 1883. By 1890 plants were in operation at Colorado Springs, Colo., Helena, Montana, and Santa Rosa and Los Angeles, Cal. Greeley, Colo., Hastings, Neb., and Trinidad, Colo., followed very soon after. For a time it looked as if sewage farming were to be the standard method of sewage disposal for the western states. Carelessness in operation leading to local nuisances, and a growing feeling that sewage was less satisfactory than water as an irrigating medium, have however led to the abandonment of many of the farms in operation twenty years ago. Both Cheyenne and Los Angeles have abandoned plants of this type; and, as Metcalf and Eddy (1916) have pointed out, many so-called sewage farms are really such only in name. Thus at Hastings the attempt to raise crops has been abandoned while at Salt Lake City " The city leases all the sewage to a farmer, who makes very little use of it. The period of his lease is for 25 years from Nov. 30, 1903; the consideration, $1. The sewage not used flows through an open outlet ditch to Great Salt Lake."

California seems to be the one state where broad irrigation

flourishes. According to Metcalf and Eddy 35 communities in California employed this method of treatment in 1914. In some cases the land is owned or leased and operated by the town; in other instances the sewage is distributed on private land, with or without the payment of a bonus. The crops cultivated are those usual in the respective localities — peas, beans, tomatoes, corn, cabbages, turnips, grass, alfalfa and fruit trees. In the more recent plants, as at Santa Rosa, Pomona and Fresno, Cal., a septic tank has been installed for preliminary treatment of the sewage. In many other cases it must be acknowledged that neither the construction nor the operation of the farms is especially calculated to secure prompt and inoffensive oxidation of organic matter. " The sanitary question of disposing of the sewage seems to be quite incidental " (Fuller, 1905).

The Pasadena sewage farm may be taken as a type of broad irrigation at its best, from the economic standpoint. Three hundred acres of land were purchased by the city in 1887, and after a long period of litigation in regard to rights of way for an outfall sewer, the application of sewage was at last begun in 1892. The area was gradually increased to 518 acres in 1914; and a septic tank for the preliminary treatment of the sewage was installed in 1910. The farm was about four miles from the city and the soil a good sandy loam. The sewage was at first distributed mainly by ditches but a series of concrete pipes were laid in 1910. The crops have consisted of barley and wheat, hay, pumpkins, corn, alfalfa and English walnuts. The production of alfalfa was abandoned for a time since, with this matted low-growing crop, which cannot be cultivated, the sewage solids collected in the upper part of the field and caused offensive conditions. Since the installation of the septic tank to remove these solid materials alfalfa has been grown again and in recent years, alfalfa, walnuts, oat hay and oranges have been the principal crops. The following details of operation are taken from the Town Report for 1903–04 (Pasadena, 1904).

" Whether being used in the groves or in the open fields, the sewage is not allowed to run upon any one area longer than from four to ten days at a time, after which period it is turned upon another area, and as soon as the area upon which it had been running is sufficiently dry to be worked, which is usually within two or three days, it is thoroughly cultivated or plowed. Fre-

FIG. 63. Cornfield on the Pasadena Sewage Farm.

FIG. 64. Walnuts on the Pasadena Sewage Farm.

quently the top surface is plowed under, but this is not done after every period of irrigation, as only thorough stirring with a cultivator is necessary. The number of days that the sewage can be allowed to run upon a given area at any one period depends upon the weather and atmospheric conditions. If the weather is hot and dry, the period reaches its maximum, but if the weather is damp and the atmospheric pressure low, when such odors as are present cannot rise readily but are held down close to the surface of the ground, the period is the minimum."

The average flow of sewage treated on the Pasadena farm in 1903–04 was 840,000 gallons per day. On the 300 acres of land then in use, this was equivalent to a rate of 2800 gallons per acre.

The operating expenses of the farm for this year were $6310.91, mainly for labor. The revenues amounted to $11,643.57, of which $7847.29 came from the sale of walnuts. The land originally purchased in 1887 cost $125 an acre, or about $40,000 in all. In 1904 one hundred and sixty acres more were purchased at $150 an acre. Interest at $3\frac{1}{2}$ per cent on the cost of the three hundred acres in use in 1903–04 would amount to $1400. Even taking this into account, the Pasadena farm was clearly a profitable undertaking.

Metcalf and Eddy (1916) cite the farm at Pomona, Cal., as being self-sustaining; and at Fresno, Cal., 794 acres of irrigated land were leased to private parties in 1908 for a ten year period at a rental of $9000 a year, more than enough to pay the interest on the bonds issued to pay for the disposal works (Eng. Rec. 1908).

Another very successful sewage irrigation project has been carried out by the city of San Antonio, Tex. (Bartlett, 1916). The sewage from a population of 120,000, averaging 13,500,000 gallons daily in 1916, is carried by a 72-inch sewer and an open ditch to the broad shallow basin of Mitchell's Lake where it is used by a private company for irrigation. The total area of irrigated land in 1916 was about 1600 acres. Foodstuffs, sorghum, alfalfa and cotton have at times been grown in the area, but the more recent policy of the company has been to convert the farms into Johnson-grass pastures and raise cattle upon them. During the winter months the sewage is impounded in the lake itself, a shallow body of water about 900 acres in area. Of 15,000 acre-feet of sewage annually discharged, 8000 are

applied to the irrigated lands (at a rate of about 10,000 gallons per acre per day) and 6000 acre-feet evaporate from lake and ditches. The lake receives in addition to the sewage some 400–800 acre-feet of storm water. It is therefore increasing somewhat in size and in wet seasons some of the sewage at times of storm is wasted into the Medina River. No serious nuisance appears to have been created and the lake, even with such a high concentration of sewage, is characterized by green algal growth rather than by the fungous growths of putrefactive decomposition.

It should be noted in all these cases that success depends on the water value of the sewage rather than on its fertilizing constituents. This is clearly shown by the effort to remove these latter constituents so far as possible by preliminary treatment. Fuller (1912) estimates the total value of the sewage as used for irrigation in our western states at not over $1.50 per million gallons as delivered at the farm. Pasadena, so long one of the most successful examples of sewage farming began in Feb., 1917, experiments with the activated-sludge process of treatment as a possible preliminary to irrigation. Such a procedure as this would of course conserve the nitrogen of the sewage in a highly available form while at the same time preventing clogging or local nuisances; but it would represent a complete abandonment of the conception of irrigation as a *purification* process.

The General Outlook for Sewage Farming. The availability of broad irrigation, as a practical method of sewage treatment, obviously depends to a high degree on varying local conditions. In dry countries, where every drop of water is precious, as in parts of California it is likely to remain a useful method of sewage treatment. In India the rice fields at Madras and elsewhere are irrigated with sewage with marked success. On the other hand, it seems quite as certain that sewage farming on heavy lands is a mistake. The English communities, which have clung to sewage farming under adverse natural conditions, have demonstrated that it may be a failure, and a costly one. Between these two extremes are cases in which the conclusion is less clear. With fair soil and not too heavy rainfall, broad irrigation may be operated satisfactorily by cities having at their doors large areas of cheap and infertile sandy soil. Its economic value, then, depends upon a number of minor variables. The

cost of land, the cost of labor, the available markets, and above all, the skillful management devoted to the farms, chiefly control the final result. Where all these conditions are favorable, broad irrigation may offer an economical method of sewage disposal, as demonstrated at Aldershot and at Berlin. Where all or any of these conditions are against the process, its success becomes more dubious. In England the general tendency is toward a decrease rather than an increase in the number of sewage farms. Calmette (1907), after a careful discussion of conditions on the Paris farms, concludes that the practice of broad irrigation is likely to be more and more restricted and ultimately abandoned, and Dunbar (1908) believes that even at Berlin artificial filters will one day take the place of sewage farms. Stroganov, the director of the Moscow farms, looks forward to their abandonment.

In any case, where sewage farms are to be maintained, the danger that sanitary efficiency may be sacrificed to economic success must be carefully guarded against. There is an inevitable antithesis between the agricultural and the sanitary requirements of broad irrigation, and only a large plant can reasonably hope to hold a superintendent competent to harmonize both aspects of the problem.

In the eastern United States natural conditions of soil and rainfall are generally more favorable to sewage farming than those which prevail in England. Economic and political conditions, however, are against the process. The excessive water consumption in this country makes the per capita flow of sewage two or three times as high as it is in Europe. The labor charges in the United States would be twice as high as in England; and municipal politics seriously militate against efficiency of operation, though this latter objection is a temporary rather than a permanent one. Sewage farming is, however, not likely to be one of the future activities of the American city, except in the arid regions.

CHAPTER IX

DISPOSAL OF SEWAGE BY INTERMITTENT FILTRATION THROUGH SAND

Early Studies of the Principles of Bacterial Purification. Dilution in water and broad irrigation on land are the two primitive and natural methods of sewage disposal. Both systems were at first developed from a purely empirical standpoint, and without any idea that the fundamental process, in either case, was the oxidation of organic matter under the influence of bacterial life. A few investigators, however, grasped the essential principles involved at an early date in the history of the art. Thus, about 1865, Alexander Mueller, City Chemist of Berlin, described the purification of sewage as a process of digestion and mineralization carried out by minute animal and vegetable organisms. In 1878 he took out a patent for a " process for the disinfection, purification and utilization of sewage by the scientific cultivation of yeastlike organisms." In 1877 the fact that the purification of sewage is due to living organisms was demonstrated by Schloesing and Müntz in a series of experiments, in which it was shown that nitrification did not occur in soils sterilized by heat or chloroform (Schloesing and Müntz, 1877). Warington, in England, communicated to the Society of Arts, in 1882, a paper in which he pointed out that dilute solutions of ammonium salts or of urine would not nitrify when sterilized by boiling and supplied with air filtered through cotton. If, however, a small particle of fresh soil was added, active nitrification would set in. He further found that this process went on best in the dark, in the presence of an alkaline base, such as lime, and at a temperature of 5°–50° C. He adds, " Though, however, porosity is by no means essential to the nitrifying power of a soil, it is undoubtedly a condition having a very favorable influence on the rapidity of the process; a porous soil of open texture will present an immense surface, covered with oxidizing organisms, and generally well supplied with the air requisite for the discharge of their functions."

Beginnings of Intermittent Filtration in England. The practical development of sewage purification, along intensive lines, and by the application of scientific principles, had begun even earlier than this with a series of significant experiments carried out by Sir Edward Frankland in connection with the first report of the Rivers Pollution Commission of Great Britain. In the early days of irrigation it was generally thought that the green plants played an essential part in the purification process. When the experts of the Commission visited Ealing and Chorley, they found that sewage was being treated with more or less success on uncropped land. This suggested the importance of a careful study of the purifying process, and Frankland at once began a series of experiments in his laboratory at London. Glass cylinders six feet high and ten inches in diameter were filled with various filtering materials, — gravel, sand, loam and peat, — and London sewage was applied in various amounts, twice daily, for a period of four months. The results, on the whole, were highly satisfactory, and the Commission concluded that the sewage of 1000 persons could be treated on an acre of properly prepared sandy soil. Sewage was filtered, in Frankland's experiments, both upward and downward through various soils at different rates, and it was shown that a good effluent could be obtained by downward filtration through coarse gravel at a rate of 80,000 gallons per acre per day, while upward filtration produced only a foul and turbid effluent. Doubling the rate interfered with the purification, and it was noted that a resting or aerating period between the applications of sewage was a necessity. The principle of the process as a chemical oxidation of organic matter to water, carbon dioxide and nitrates was clearly recognized, as well as the practical necessity for intermittency in operation. The cycles in the life of a sewage filter were picturesquely described in the following passage: " The conclusions arrived at may be thus summarized: Sewage traversing a porous and finely divided soil undergoes a process to some extent analogous to that experienced by blood in passing through the lungs in the act of breathing. A field of porous soil, irrigated intermittently, virtually performs an act of respiration, copying on an enormous scale the lung action of a breathing animal, for it is alternately receiving and expiring air, and thus dealing as an oxidizing agent with the filthy fluid which is passing through it. The action of

the earth as a means of filtration must not be considered as merely mechanical; it is chemical, for the results of filtration properly conducted are the oxidation, and thereby the transformation, of the offensive organic substances, in solution in the sewage, into fertilizing matters, which remain in the soil, and into certain harmless inorganic salts, which pass off in the effluent water" (Rivers Pollution Commission, 1870).

These researches of the Royal Commission indicated that with suitable soil the process of broad irrigation might be made intensive, the growing of crops being subordinated to the treatment of sewage at a more rapid rate. This principle was quickly applied on a practical scale by J. Bailey-Denton, who constructed an intermittent filter at Merthyr Tydvil, Wales, in 1871. About twenty acres of gravelly land, overlayed with loam, were laid out in four beds, carefully underdrained at a depth of five to seven feet. The surface of the beds was furrowed and cropped, but they were operated in accordance with Frankland's recommendations as true intermittent filters. The sewage was applied to each bed for six hours out of the twenty-four and the net rate was about 60,000 gallons per acre per day. This rate was later reduced to 16,000 gallons per acre by the addition of more land, and the plant has since been operated more in the fashion of an irrigation area (Harvey, 1908). The original plant worked admirably, however, and was of importance as a demonstration of Frankland's process on a practical scale (Bailey-Denton, 1882).

Bailey-Denton installed other plants in England and Scotland upon the same principles; but as a rule English engineers were not impressed with Frankland's experiments. The conception of intermittent filtration was merged and lost in the prevalent practice of broad irrigation and the intensive biological purification of sewage made no appreciable headway. As in so many other instances, the theoretical comprehension of a process was by no means the same thing as its demonstration in such a convincing form as to insure general acceptance. The real introduction into practice, of modern scientific methods of sewage treatment remained for American investigators. English sanitarians have generously recognized that "it was primarily due to the Massachusetts State Board of Health, who began their investigations in November, 1887, and have continued them ever

since, that the bacterial treatment of sewage has been forced on public attention" (Watson, 1903).

The Lawrence Experiment Station of the Massachusetts State Board of Health. By the year 1880 the eastern part of the state of Massachusetts had become so thickly settled that the problems of stream pollution and sewage disposal began to press for solution. The same conditions which had been met in England twenty years before were beginning to be faced in the United States. Accordingly, in 1881 and 1884, special commissions were appointed to consider the pollution and protection of the streams in the vicinity of Boston. The second of these bodies made a report in 1886 which contained a review of sewage disposal practice in England and on the Continent and an earnest recommendation for a permanent commission intrusted with the duty of protecting the purity of inland waters. The paragraph in which the functions and the policies of such a body are outlined is so admirably expressed that it may well be quoted in full:

"Let these guardians of inland waters be charged to acquaint themselves with the actual condition of all waters within the state as respects their pollution or purity, and to inform themselves particularly as to the relation which that condition bears to the health and well-being of any part of the people of the commonwealth. Let them do away, as far as possible, with all remediable pollution, and use every means in their power to prevent further vitiation. Let them make it their business to advise and assist cities or towns desiring a supply of water or a system of sewerage. They shall put themselves at the disposal of manufacturers and others using rivers, streams or ponds, or in any way misusing them, to suggest the best means of minimizing the amount of dirt in their effluent, and to experiment upon methods of reducing or avoiding pollution. They shall warn the persistent violator of all reasonable regulation in the management of water, of the consequences of his acts. In a word, it shall be their especial function to guard the public interest and the public health in its relation with water, whether pure or defiled, with the ultimate hope, which must never be abandoned, that sooner or later ways may be found to redeem and preserve all the waters of the State" (Massachusetts, 1886).

In accordance with these recommendations, the Legislature of 1886 reorganized the State Board of Health and charged it with the duties outlined by the commission, namely, with the advice of cities and towns, corporations and individuals, as to water

supply and sewage disposal, and ordered it to collect information and conduct experiments on the purification of sewage. The board promptly presented plans for extensive studies of the fundamental principles involved in sewage treatment and asked and obtained a considerable appropriation for its work.

Hiram F. Mills, a distinguished hydraulic engineer and a member of the board, organized the investigation, with the assistance of Professors T. M. Drown and W. T. Sedgwick, both of the Massachusetts Institute of Technology, as chemist and biologist,

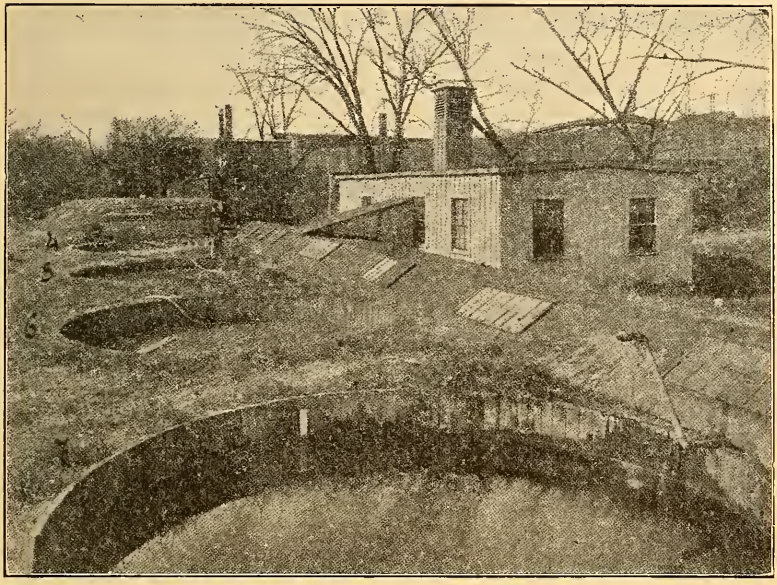

Fig. 65. Experimental Filters at the Lawrence Experiment Station (copied by permission from Henneking, 1909).

respectively. An experiment station (Fig. 65) was fitted up in 1887 on the bank of the Merrimac River at Lawrence, under the immediate charge of Allen Hazen. Ten circular cypress tanks, 17 feet in diameter and 6 feet deep, were filled with various filtering materials — sand, gravel, peat, river slit, loam, garden soil, and clay — and dosed with sewage pumped from the city sewer. From most of the filters, effluents of a high degree of purity were obtained; and the tanks themselves remained clean and sweet, confounding the bystanders. who predicted that " in a

fortnight, at the latest, the filters would become clogged and foul, and the whole neighborhood pestilential" (Sedgwick, 1902).

One condition was found essential to success, — the application of the sewage intermittently, in small doses, so that the air supply necessary for its oxidation should be available. With porous sand and gravel the proper balance of sewage and oxygen was easily maintained. A layer of sewage an inch in depth applied each day to such a bed of sand spreads itself through a layer of sand about nine inches in depth and there rests in thin films on the surfaces of the sand grains, in intimate contact with about twice its own volume of air — until forced out by a succeeding dose. Under these conditions the sewage is rapidly converted into a clear and well-nitrified effluent. With more impervious soils it appeared almost impossible to preserve this oxygen supply, since capillarity prevented them from ever draining dry. Thus with peat and garden soil, even when operated at very low rates, clogging occurred and nitrification failed. All the coarse grain filters, however, showed good purification at rates of from 20,000 to 100,000 gallons per acre per day, the chemical quality of the effluents being equal in many cases to that of well waters in use in the city of Lawrence.

The true nature of sewage purification as a bacterial oxidation was clearly brought out in these experiments. Intermittency of application supplies the needed oxygen, and any fairly porous material will serve as a resting place for the active bacteria.

"The experiments with gravelstones give us the best illustration of the essential character of intermittent filtration of sewage. In these without straining the sewage sufficiently to remove even the coarser suspended particles, the slow movement of the liquid in thin films over the surface of the stones, with air in contact, caused to be removed for some months 97 per cent of the organic nitrogenous matter, a large part of which was in solution, as well as 99 per cent of the bacteria, which were of course in suspension, and enabled these organic matters to be oxidized or burned, so that there remained in the effluent but 3 per cent of the decomposable organic matter of the sewage, the remainder being converted into harmless mineral matter.

"The mechanical separation of any part of the sewage by straining through sand is but an incident, which, under some conditions, favorably modifies the result; but the essential conditions are very slow motion of very thin films of liquid over the

surface of particles having spaces between them sufficient to allow air to be continually in contact with the films of liquid.

"With these conditions it is essential that certain bacteria should be present to aid in the process of nitrification. These, we have found, come in the sewage at all times of the year, and the conditions just mentioned appear to be most favorable for their efficient action" (Mills, 1890).

The Lawrence work had extended over a period of two full years when the first report was published in 1890. The experimental filters were of sufficient size ($\frac{1}{300}$ of an acre each) to warrant the application of their results on a practical scale. They had been operated out of doors, in all weathers, and at all seasons. The effect of these early experiments was, therefore, to demonstrate beyond a doubt that intermittent filtration was a valuable working method for the purification of sewage.

One of the first results of the Lawrence experiments was the construction at Framingham, under the supervision of Simpson C. Heald, a young Massachusetts sanitary engineer, of an intermittent filtration area for the treatment of the town sewage. The sewage, which in times of dry-weather flow amounted to 650,000 gallons per day, was run to two reservoirs (each with a capacity of 431,000 gallons) and then pumped to a filtration area of about twenty acres. This plant was constructed in 1889, and has been operated ever since 1890 with excellent results. Other communities quickly followed suit. Gardner and Marlboro constructed filtration plants in 1891 and at the same time the first filter in Connecticut was built at Meriden.

General Principles of Intermittent Filtration. Intermittent filtration differs from broad irrigation as a controlled scientific process differs from a merely empirical one. Instead of pouring sewage over any convenient plot of land, specially selected areas of sand or gravel of proper fineness and evenness are used. Instead of allowing the sewage to find its way over or through the land as best it may, the beds are carefully underdrained so that it shall filter through a thickness of four or five feet of aerated sand. The application of the sewage is so regulated by intermittent dosing that the bed shall never become waterlogged so as to cut off its air supply. As compared with broad irrigation the volume of sewage treated per unit area was increased tenfold by the regulated intermittent process (50,000–100,000 gallons per acre per day against 5000–10,000).

The most important result of the Massachusetts experiments was, however, not merely the discovery that intermittent filtration offered an excellent method of sewage disposal for communities having available sand deposits in their neighborhood; it was rather the clearing up and making definite of the essential nature of the sewage purification process itself. This process had been more or less clearly understood by Mueller, Schloesing and Müntz, Warington and Frankland, but its details were elaborated and its practical importance convincingly proved for the first time by the Lawrence workers. Their greatest contribution to the art was the demonstration of the fundamental fact that sewage purification is a slow burning or combustion of organic matter, carried out by living micro-organisms growing on the surface of a porous substratum and working only in the presence of abundance of oxygen. The general principles established at Lawrence underlie all newer processes of sewage treatment of any type whatsoever. The contact bed, the trickling filter and the activated sludge process are modified and improved devices for applying the laws worked out in their elementary form in the little testing laboratory on the shore of the Merrimac.

The Chemistry of Intermittent Filtration. The chemical changes which take place in the sand filter are, so far as we now understand them, relatively simple ones. They consist in essence of a more or less direct oxidation of organic matter, the nitrogen finally appearing in the form of nitric acid and the carbon and hydrogen as carbonic acid and water. There appears, however, always to be an intermediate stage in which nitrous acid is formed from the organic nitrogen, to be later oxidized to the nitric form. The nitrous and nitric acids are present, of course, in combination with the alkali metals and the alkaline earths, while the carbonic acid is partly combined and partly escapes as gas.

Measured by the ordinary methods of sanitary chemical analysis, these changes are manifested by the disappearance of ammonia and albuminoid nitrogen and the formation of nitrites and nitrates. Under favorable conditions the transformation of nitrogen to the mineral form may be almost quantitative. The classic experiment of Scott-Moncrieff at Ashtead, in 1898, furnishes an excellent illustration of the nature of the process, although his filtering material was of coke, like that used in trickling filters,

and was not therefore exactly comparable with sand. He constructed a series of nine trays of 1-inch coke, each 2 by 7 feet by 7 inches deep, arranged one over the other, with a space of 2 inches between each pair. The effluent from a " cultivation tank " was discharged on the upper tray by tipping buckets at a rate of 1,300,000 gallons per acre per day (140,000 on the whole area of nine trays), and its passage through the series occupied from eight to ten minutes. The results of the gradual purification are indicated in the table on page 242, and the steady decrease of organic matter, with a corresponding increase of nitrates and a temporary formation of nitrites, will be observed. There was a total loss of a quarter or more of the nitrogen in this experiment; but in some similar work by Rolants and Gallemand (1901) it was shown that 297.9 mg. of nitrogen out of 301.8 mg. applied could be recovered at the end of the process in the nitric form.

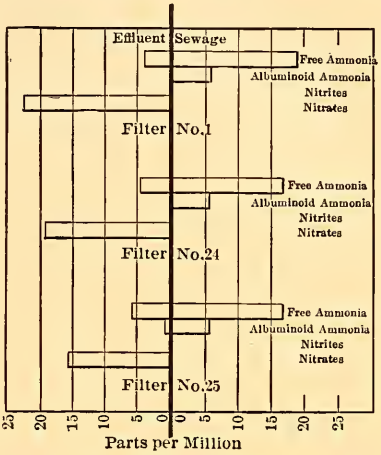

FIG. 66. Nitrogen Changes in Intermittent Filtration at the Technology Experiment Station.

The general course of the changes in the passage of sewage through a sand filter is indicated graphically in Fig. 66, from results obtained at the Sewage Experiment Station of the Massachusetts Institute of Technology (Winslow and Phelps, 1906).

The Bacteriology of Intermittent Filtration. Pasteur expressed the conviction in 1862 that the process of nitrification would eventually be shown to be due to the activity of living microorganisms. This suggestion was verified by Schloesing and Müntz in 1877. They prepared experimental filters of quartz sand mixed with crushed limestone and passed sewage through them for a considerable time. At first there was no change, but gradually nitrification set in and the filters finally became very active. When exposed to the vapors of chloroform the process stopped, and after the removal of the chloroform nitrification soon began again, if the filter was seeded with earth from a

TABLE LII

RESULTS OF TRICKLING FILTRATION THROUGH SCOTT-MONCRIEFF'S TRAYS

Parts per million. (Scott-Moncrieff, 1899.)

Effluents of —	Nitrogen as —				
	Ammonia N.	Albuminoid N.	Nitrites.	Nitrates.	Oxygen consumed in 4 hours at 80° F.
Cultivation tank	103	12.3	0	1.2	98.4
First tray	86.5	10.3	9.9	1	66.9
Second tray	74.2	8.2	9	4.8	57.7
Third tray	41.2	4.9	7.8	18.7	44.9
Fourth tray	33	2.9	6.6	27.6	17.3
Fifth tray	12.4	1.2	4.8	46.8	12.8
Sixth tray	14.4	2.9	5.1	44.2	15
Seventh tray	2.9	2.5	0	66	7.6
Eighth tray	1.7	5.3	0	73.2	4
Ninth tray	2.1	4.9	Slight tr.	90	5.9

nitrifying field. Heating to 100° was also shown by these investigators to stop all action, and it was thus proven that living organisms of some sort were the cause of this particular fermentation (Schloesing and Müntz, 1877). Warington and the Franklands and many other observers extended and confirmed these results; but for a long time all attempts to isolate the particular organisms concerned seemed doomed to failure. Finally the Franklands, by the dilution method, succeeded in isolating from a nitrifying solution a short, stout bacillus which formed nitrites actively but which refused to grow on gelatin media (Frankland, 1890). In the same year, Winogradsky (1890) independently discovered the same organism, which he called *Nitrosomonas*, growing it in solutions containing ammonium sulphate, potassium phosphate and basic magnesium carbonate without organic material. In the next year Winogradsky (1891) showed that the change from nitrous to nitric acid is dependent upon the presence of another peculiar group of organisms (*Nitrobacter*), which is also difficult to cultivate on ordinary media and which, working in symbiosis with *Nitrosomonas*, is able to carry on the complete nitrifying change. In this year, too, he successfully cultivated these organisms on solid media made up of inorganic constituents stiffened with silica jelly. Richards and Jordan (1890) in the United States, and Warington (1891) in England, promptly confirmed

these results. An important series of investigations of the nitrifying organisms was later reported by Winogradsky and his colleagues (Winogradsky and Omelianski, 1899, and Omelianski, 1899), from which it appeared that the *Nitrosomonas* acts only upon ammonia nitrogen and is unable to attack more complex organic bodies. The failure to find the nitrifying bacteria prior to 1890 was due simply to the unsuitability of the rich organic media employed. In recent years they have been isolated and studied by a large number of workers, of whom the most important are perhaps Schultz-Schultzenstein (1903), Boullanger and Massol (1903), Wimmer (1904), Calmette (1905) and Chick (1906).

The nitrite organisms, grouped under the name *Nitrosomonas*, differ more or less in soils from various parts of the world. They are all, however, oval bacilli which grow in compact zoöglea masses; some forms have a motile stage and others have not. In the process of their growth they apparently utilize the energy of the ammonia compounds to break up the carbonic acid molecule, from which they obtain their supply of carbon. Substances like urea, asparagin and egg-albumin check their activity, and even food material as close to ammonia as the amines cannot be attacked. It is clear, therefore, that in nature the *Nitrosomonas* organisms are dependent on a sort of symbiosis with the ordinary putrefactive bacteria which set free their food supply, ammonia, from its organic compounds. It appears probable, too, that the nitrite organisms only thrive well in the presence of a porous inorganic substratum on which they can form their zoöglea films. Stevens has shown, using both pure and mixed cultures, that nitrification is far more active in a partially dry soil than in a saturated soil or liquid (Stevens and Withers, 1909–1910).

The nitrate formers of the genus *Nitrobacter* are smaller oval rods, capsulated, and responding with difficulty to ordinary bacterial stains. They grow on agar media better than the nitrite formers, though with extreme slowness; but a very small amount of ammonia nitrogen checks their development. The nitrate formers are therefore dependent upon the nitrite formers for protection against ammonia, as the nitrite formers are dependent on the putrefactive bacteria for the formation of ammonia from more complex substances. An interesting point, brought out by Beddoes (1899) and Adeney (Letts and Adeney,

1908), is the favorable effect exerted upon the activity of *Nitrobacter* by the presence of humus-like bodies.

The main practical requirements for the whole process of nitrification, as worked out on the basis of these bacteriological studies, may be stated somewhat as follows: A porous substratum must be provided for the growth of the active organisms. They must have an abundant supply of oxygen for their specific fermentations. The applied sewage must not be too strong; *Nitrosomonas* development is checked by the presence of 0.05 per cent of ammonia nitrogen. An alkaline base must be provided (as agriculturists have known since the days of Varro), to unite with the nitric acid produced, since free acids, in a strength of 0.5 per cent, quickly stop the nitrifying process. As a rule, however, the alkalinity of ordinary domestic sewage is sufficient for this purpose. Nor is the particular base present an indifferent matter; the nitrites and nitrates of calcium and magnesium are less harmful than the corresponding salts of sodium and potassium; 0.1–0.5 per cent of sodium nitrate retards nitrosification as against 1 per cent of calcium nitrate. In any case, free drainage to remove the end products of the reaction must be provided. Finally, a marked excess of alkali may be detrimental. Burton sewage containing 100 parts of free lime would not nitrify till it was neutralized (Barwise, 1904). Rideal also notes a harmful effect due to carbon dioxide, and Letts believes that sodium chloride from sea water hinders the formation of nitrates at Belfast (R. S. C., 1902). The optimum temperature for the process of nitrification lies between 28° and 37° C. The *Nitrosomonas* forms are destroyed at 45° and the *Nitrobacter* group at 55°.

A sewage filter is really a device for cultivating the *Nitrosomonas* and *Nitrobacter* organisms under the conditions most favorable to their maximum activity. Like any other biological mechanism, it requires time to bring it to perfection. In the original Lawrence experiments it was found that nitrification did not start in new filters during the cold weather of winter. The delicacy of the reactions involved is shown very clearly in the trickling filter which responds even more readily to external changes. Fig. 88, in Chapter XI, is taken from the report of the Technology experiments with two eight-foot beds of broken stone. For the first six months of the life of the beds no impor-

tant nitrification occurred. With the warm weather of spring, nitrites began to appear, reaching a maximum in September; and as the nitrites reached their height the nitrate formation began, and continued with increasing vigor during the whole of the following winter.

Construction of Intermittent Filters. The construction of intermittent sand filters in regions like Massachusetts is extremely simple. This particular part of the United States is covered with

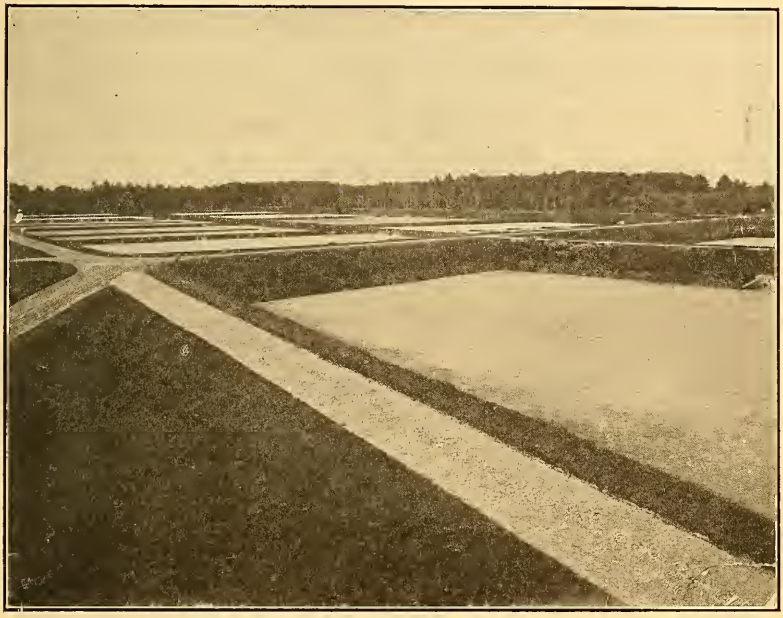

FIG. 67. General View of Intermittent Filter Beds at Brockton, Mass. (courtesy of G. E. Bolling.)

deposits of glacial drift, so that large areas of fairly level sandy soil are of very common occurrence, and all that it is necessary to do is to strip off the surface soil, usually not more than 1 foot in depth, to level off the sand area, divide it into beds by embankments made of the strippings, remove from the sand beds so made, any pockets of clay or quicksand, and underdrain the beds — best by digging trenches 4 feet to 6 feet deep, about 40 feet apart, the first being 25 feet from the edge of each bed, and placing in these trenches vitrified clay pipes, laid with open

joints. These drains are connected with main drains, placed in the embankments between the beds, and the sewage is brought to the plant by gravity or by pumping, and by various methods distributed upon the surface of the beds.

Where deep layers of porous sandy soil are available it may not be necessary to lay any underdrains at all, the filtered sewage being allowed to mingle with the ground water and find its way along with it to ditches in the lower portions of the filtration area. As a rule, however, underdrains should be provided. According to Metcalf and Eddy (1916) lines of underdrains should be laid from 30 to 40 feet apart at a depth of 3–4 feet below the surface at their upper end and with a slope of 6 inches or more in 100 feet. These authors recommend that the spigot ends of each pipe should be separated from the shoulders of the adjacent bell by a space of $\frac{3}{8}$ inch and that the upper part of the bell should be broken off so as to allow full access to the drainage liquid. The underdrain should be surrounded by screened gravel or broken stone 2-inch material immediately around the drain with a layer of 1-inch material next and finally a layer of $\frac{1}{4}$-inch material to support the main sand layer.

The general plan of an intermittent filtration area is well illustrated by the plant at Brockton, Mass. (see Fig. 68). This disposal area is located in the southwesterly corner of the city, adjoining the towns of Easton and West Bridgewater. The original plant comprised 23 beds; seven new ones were added in 1905, and seven more in 1908. The provision of a laboratory building where the operation of the beds is controlled by frequent and regular analyses may be noted as a most important feature of the plant. At small plants, to reduce expense, such a laboratory can also be developed, as at Brockton, into a general municipal laboratory.

The average area of the beds at Brockton is about one acre each; but shape and size are varied to suit the configuration of the ground. A large peat hole rendered about two acres of the area unfit for use. The beds were prepared for receiving sewage by the removal of all of the loam from the surface. The sand is stratified, but the different strata are not separated, in most cases, by a distinct line of stratification, and the material as a rule is coarse and porous. In general, the underdrains are laid about 60 feet apart and at a depth of 7 to 9 feet. They discharge

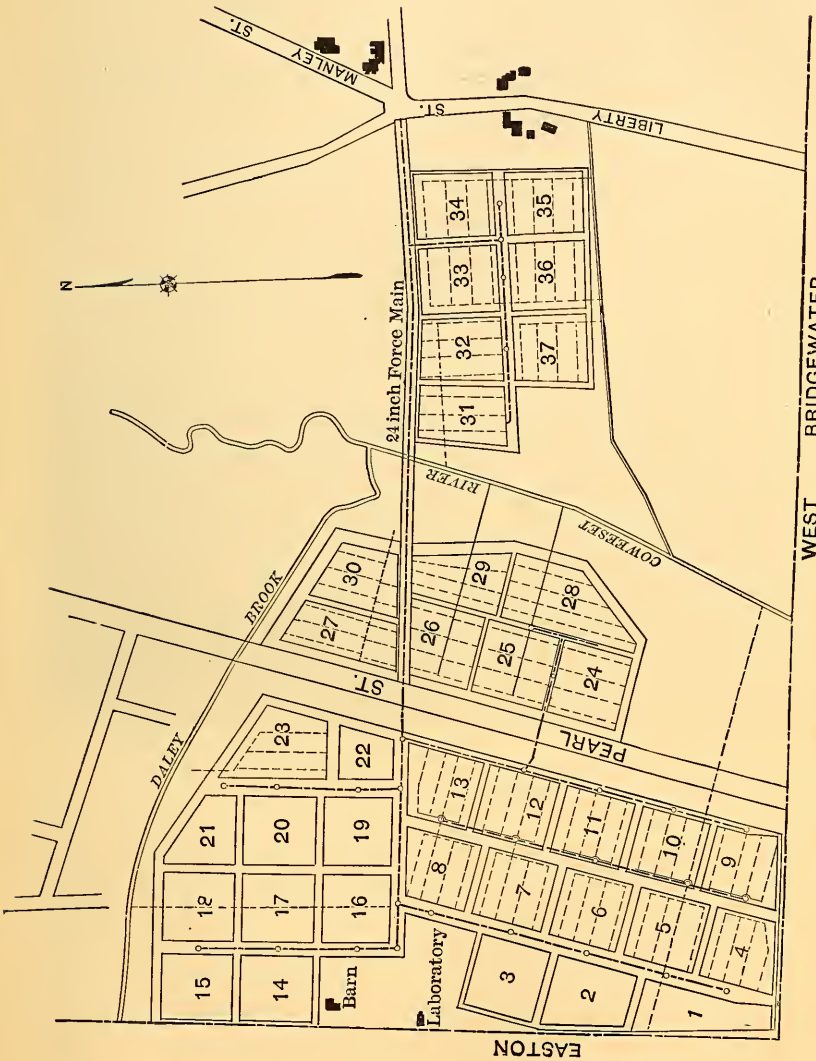

FIG. 68. Plan of Brockton Filter Beds (courtesy of G. E. Bolling).

into main drains, varying in diameter from 8 to 15 inches, which carry the effluent into the Coweeset River, or into a small tributary of the river.

The sewage is distributed on the surface of the beds by carriers laid across the bed from the center of one side. They are simply flat-bottomed sluiceways reducing from a width of 5 feet at the inlet to 1 foot at the extreme end by a series of offsets of 6 inches on each side. At each offset is an opening controlled by slanting

FIG. 69. Furrows and Sewage Distributors at Brockton, Mass. (courtesy of G. E. Bolling).

wooden gates, by which the amount of sewage discharged on any portion of the bed may be controlled. As originally constructed the carriers were made of wood, but these have been replaced by carriers with concrete bottoms 5 inches deep, and having the angle irons to which the wooden sides are fastened embedded in the concrete. The initial cost of such carriers is less than when made entirely of wood and they are much more permanent. A view of the bed with its carriers is shown in Fig. 69.

Selection of Suitable Sand for Intermittent Filters. Where suitable sand beds are not found *in situ* the soil must be excavated

to a depth of 3 to 5 feet and replaced by suitable clean, coarse sand taken from some neighboring knoll. Outside of the glacial drift area it may often be necessary to bring suitable sand from a considerable distance. The proximity of an adequate supply of sand often becomes a controlling factor in determining the availability of this method of sewage treatment. The construction of an artificial sand bed has, however, a compensating advantage in the fact that the beds under these conditions can be built of the very best grade of material. The character of a particular deposit is determined, as in the study of sand to be used for water filtration (Hazen, 1893, Hazen, 1900). Test pits are dug in various parts of the sand bed and representative samples of one hundred grams or more are collected for examination. Each sample is then passed, with vigorous shaking, through a series of sieves, of mesh ranging from 0.1 mm. to 5 mm., or thereabouts. The sand remaining on each sieve is weighed and a curve plotted, using the size of each mesh, in terms of millimeters, as ordinate and the per cent of the whole sample by weight which passed that sieve, as abscissa.

It was shown in the Lawrence experiments that the finer particles of sand are the ones which by packing between the others actually determine the rate and efficiency of filtration. The character by which a sand is judged, called its "effective size," is that size, than which 10 per cent by weight of the particles are smaller. This size can of course be read directly from the curve of weights of successive screenings plotted as described above.

For sewage treatment the limits of effective size may vary much more widely than in water filtration. If this value is too great, however (over 0.40 mm.), filtration will be likely to be too rapid for efficient purification. If on the other hand it is below 0.20 mm. filtration will be slow. Filters of the latter type will of course operate successfully if the dosage of sewage be very small. A sand with an effective size between 0.25 and 0.35 mm. should be selected if possible.

Within the limits cited the rate of filtration will in general vary directly with the effective size. Thus at the Lawrence Experiment Station a sand with an effective size of 0.04 mm. treated sewage at a rate of 20,000 gallons per acre per day, while sand with an effective size of 0.48 operated at three times this rate.

Another valuable measure of the suitability of filter sand is its uniformity coefficient, which is the ratio between the mesh which would pass 60 per cent of the sand and the mesh which would pass 10 per cent of it. The nearer this figure comes to unity the more homogeneous and the more suitable is the material. Actual figures vary from 3 to 15 in natural beds and perhaps from 2 to 4 in artificially constructed ones.

It is particularly important for good results that the sand in the bed should be free from horizontal stratification of material of different size. As Fuller (1904) has pointed out: "If fine sand, loam or clay remains above coarse sand, the latter is of limited benefit for filtration, because a water seal is formed at the bottom of the fine layer, due to the liquid held in the pores by capillarity, and air is excluded from entering the sand. If coarse sand overlies fine material near the surface, clogging sooner or later takes place at the junction and air is similarly excluded."

The size of individual beds at different filtration areas varies from 0.05 of an acre up to 1 acre, the very small beds being in towns where the daily flow of sewage is small. Thus, at Leicester, Mass., where the total flow of sewage is less than 30,000 gallons per day, there are eight beds, each having a superficial area of 0.045 of an acre; while Andover, with 125,000 gallons of sewage per day, has twenty beds, each 0.18 of an acre; Marlboro, with 1,100,000 gallons of sewage, has eighteen beds, each 0.6 of an acre; Framingham, with 650,000 gallons of sewage, has eighteen beds, each a little over 1 acre in area.

Distribution of Sewage on Intermittent Filters. Sometimes the sewage is distributed to a bed from the four corners or from two quarter points on the long sides of a bed. In other cases wooden or concrete troughs are provided in which the sewage may flow to the extreme portions of the bed. Sometimes, as at Brockton, there is a single line of trough running down the center of the bed, narrowing as it proceeds, and discharging a portion of its flow at each decrease in diameter (Fig. 69). In other plants, as at Lake Forest, Ill. (Fig. 70), the troughs radiate out in a crowfoot pattern. At this plant the troughs consist of two upright sides of 2-inch plank, resting on a similar bottom plank with 3-inch square holes at the base of the sides, spaced about 2 feet apart, from which the sewage gushes out. It is well to sup-

port such troughs by vertical pipes or pieces independent of the sand to prevent uneven settling with the sand and consequent pooling and rotting.

At many of the newer filter beds a dosing tank is provided, from which, by means of automatic devices, the allotted amount of sewage can be run upon the bed in from fifteen to thirty

Fig. 70. General View of Intermittent Filters at Lake Forest, Ill. (courtesy of J. A. Alvord).

minutes; and such rapid application greatly improves distribution. Massachusetts engineers have generally adopted a dosing rate of 1 cubic foot per second for each 5000 square feet of area, and the beds are usually flooded at each dose to a depth of 1–3 inches (corresponding respectively to 30,000–90,000 gallons per acre).

Various forms of automatic dosing apparatus for intermittent filters have been developed to a considerable degree of perfection. The danger from the failure of such devices is, of course, always considerable, and they require occasional inspection by some one competent to adjust the apparatus if necessary, but a fairly accurate automatic mechanism may perhaps be considered as reliable as the average city employee. The apparatus installed

at Lake Forest by Alvord (Fig. 71) is a simple and ingenious one. A float in the dosing chamber lifts an iron ball in one of a series of wooden columns, and at a certain height the ball rolls through a trough from one column to the next, in its passage striking a catch, which opens an air valve attached to one of ten bell-siphons in the dosing chamber. Each of the siphons discharges

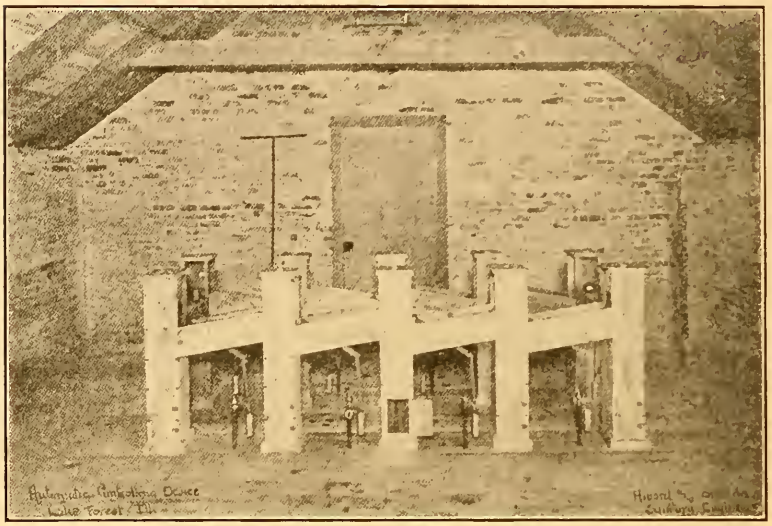

Fig. 71. Dosing Apparatus at Lake Forest, Ill. (courtesy of J. A. Alvord).

on one of the ten sand beds, which are thus dosed in rotation. The general form of siphon used with such devices is shown in Fig. 72.

Operation of Intermittent Filters. The operation of intermittent filtration plants in summer is very simple. Bacterial action is at its best, and all that is necessary is to apply the allotted dose to each bed, and occasionally to break up and remove the deposit that is left on the surface of the sand. The larger suspended matters of raw sewage are retained for the most part on the surface where they form a mat or scum which as it dries separates more or less completely from the sand and may often be rolled up and removed like a carpet. Finer solid materials, particularly when clarified sewage is treated, may penetrate into the bed so as to necessitate the removal of the

superficial layers of the sand. With a low rate of filtration and a high degree of nitrification this clogging is sometimes negligible for long periods, but some of the best Massachusetts engineers

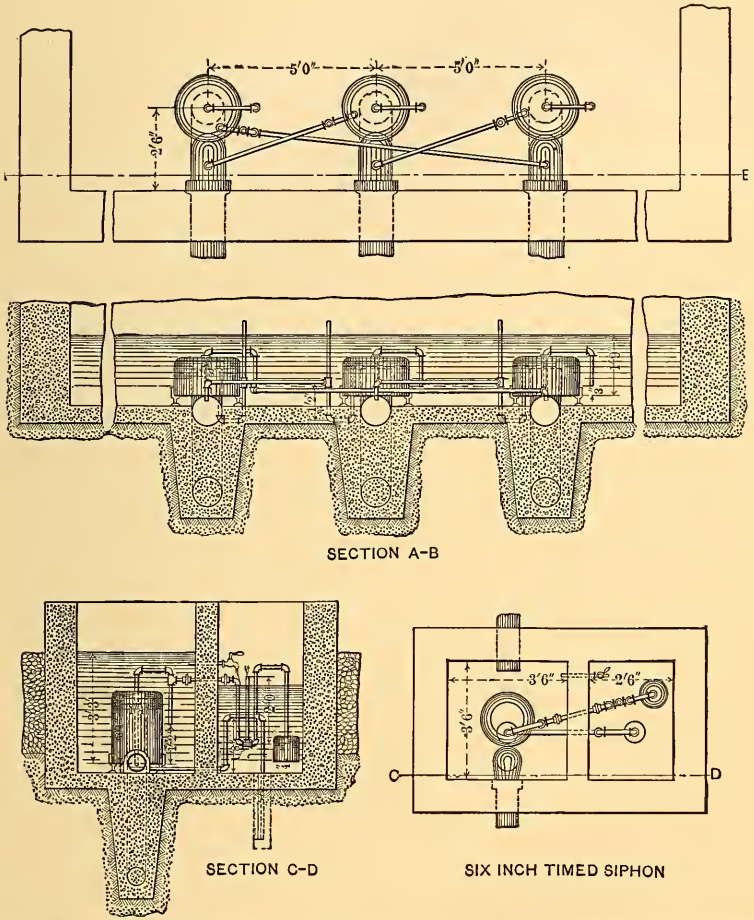

Fig. 72. Miller Automatic Flushing Apparatus for Dosing Intermittent Filters.

believe that in general it will be safest to provide for the removal of an inch of sand a year.

The surface of the intermittent filter sometimes becomes packed so hard that it must be broken up by raking or harrowing or even plowing, but these treatments should be applied with

great care and discretion since they tend to carry humus material down into the beds and to produce a condition of deep clogging which is much more serious than a superficial one.

At Brockton, Mass., with sewage containing 195 parts of suspended solids per million, and receiving practically no preliminary treatment, six to eight tons of solid matter have been removed from the beds for each million gallons of sewage filtered.

During the first years the rakings were burned on woodpiles in the open fields. The odor of the fumes was so obnoxious that, in 1898, the burning was discontinued, and the rakings were disposed of to farmers at a small price, the amount received from 1900 to 1906 for the total rakings being about $150 a year. In 1906, in order to have the rakings removed promptly, no charge was made, and in 1909 the city was fortunate enough to make a contract for five years for the disposal of all the rakings, the contractor to remove these rakings immediately, free of cost to the city, whenever a bed was cleaned. The composition of the rakings varied with the season, the moisture ranging from 5 to 50 per cent.

An average of several analyses gave the following result:

Moisture	16.22
Phosphoric acid	0.78
Potassium oxide	0.51
Nitrogen	1.45
Calcium oxide	0.30
Insoluble matter, sand, etc.	70.13

Care must of course always be taken to avoid overloading of intermittent filters. If ponding on the surface or a deterioration in the effluent indicates danger, lessened doses and longer rest periods should be given and surface clogging removed. The surface of the beds must of course be kept free from weeds during the summer season.

Winter Operation. During the long and cold winters of the northern United States, when for weeks together the temperature does not rise above zero degrees C. and the beds are covered with snow and ice from December to March, so that the surface of the beds cannot be reached, intermittent filters have to be operated with considerable judgment and care to obtain good results. The temperature of the sewage in winter as it comes to a plant

is not often below 7 degrees above zero C.; and if a comparatively large amount of sewage is applied to a bed, the frost is removed from the sand and the sewage penetrates the bed satisfactorily. There is danger, however, that before all the sewage passes down

Fig. 73. Intermittent Filter Bed at Pawtucket, R. I., in Winter.

it may become chilled and a solid layer of ice may form on the surface of the sand, which, would prevent further sewage being applied until there had been sufficient warm weather to melt the ice layer. To prevent the freezing up of the bed in this way, the

beds are prepared for winter work by furrowing (Brockton), or by making small heaps of sand at frequent intervals on the sand area (Worcester), since it has been found that, if the sewage does become chilled on a bed so prepared, the ice that is formed extends from ridge to ridge or from sand heap to sand heap, forming a natural covering over intermediate areas, and protecting them to a greater or less extent from the action of frost (Fig. 74).

The manner of working the beds is also somewhat different in winter. Two to four times as much sewage is applied in a dose

Fig. 74. Ice on Intermittent Filter Bed in Winter (courtesy of Massachusetts State Board of Health).

as in summer, and the bed is allowed to rest for a proportionally longer time. There is a certain amount of danger in this method of procedure, as without competent supervision some of the beds may not be treated with sewage often enough to maintain their nitrifying action.

Much trouble also arises in winter, from the clogging of the beds and the lack of opportunity for removing the deposit. A mat $\frac{1}{4}$ inch or more in thickness, and resembling papier-maché in appearance, forms on the top of the sand. This prevents the

sewage from passing quickly into the sand, and may, if too much sewage is applied, cause a bed to become water-logged. The amount of sewage that can be applied is thus materially reduced, and unless there is a reserve area available for winter use (or, what amounts to the same thing, the area is sufficiently large, so that the maximum dose applied in summer is much less than that which the area would purify), there is danger of a certain amount of sewage being allowed to escape untreated in winter.

Under the best conditions the effluent from a sand filter is not quite so good in winter as in summer. The nitrifying organisms are very sensitive to decreased temperature, even a difference of two or three degrees strikingly affecting their activity. Winslow and Phelps (1906) prepared the table below to show the monthly variations recorded for the experimental filters at Lawrence and for the filtration area at Brockton.

TABLE LIII

MONTHLY VARIATIONS IN SAND-FILTER EFFLUENTS AT BROCKTON AND LAWRENCE, MASS.

(Yearly average = 100.)

AMMONIA NITROGEN

	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
Brockton...	100	163	192	171	134	92	79	63	50	50	50	83
Lawrence...	213	269	204	168	84	48	16	6	8	14	32	120

ALBUMINOID NITROGEN

Brockton...	87	130	130	174	87	87	87	87	87	87	87	87
Lawrence...	175	171	150	132	92	76	67	58	62	49	62	117

NITRATES

Brockton...	85	70	67	81	90	112	118	115	133	129	110	96
Lawrence...	48	38	59	100	139	140	120	116	128	125	109	75

OXYGEN CONSUMED

Brockton...	84	132	152	178	100	84	72	68	68	84	92	108
Lawrence...	170	167	149	122	84	70	69	61	62	64	61	124

The Lawrence figures are the averages of the ratios for tanks Nos. 1, 2, 3, 4, 6 and 10, from 1895 to 1900, calculated from Clark's analyses. For Brockton the figures used cover the period from 1897 to 1904. A regular seasonal variation is indicated, the organic constituents reaching their maximum in February with the small Lawrence tanks, and in March with the large Brockton filter. The nitrates show a reciprocal curve, lowest in February. The maximum monthly deviation amounts to about 100 per cent, the worst monthly averages being twice as high as the general average in organic matter.

Nevertheless, even in winter the effluent from a well-managed filtration area is generally stable, when submitted to incubation tests.

Rate of Filtration. The rates at which intermittent sand filters can be operated with crude sewage vary from 20,000 to 100,000 gallons per acre per day, the coarseness of the filtering material and the strength of the sewage being the chief controlling factors. The tanks at the Lawrence Experiment Station have generally yielded rates of between 20,000 and 60,000 gallons per acre per day and Clark and Gage (1908) set the upper limit at 75,000 gallons per acre per day if filters are to be permanent. Fuller (1912), in an analysis of the actual operation of the various Massachusetts intermittent filters, shows that the population actually served per acre of filter has varied from 220 at Stockbridge to 2000 at Gardner. Of 15 communities cited by him 9 showed between 300 and 900 persons per acre. In general we may conclude that it is safe to allow between one and two acres of filter surface for each thousand persons. This corresponds roughly to a rate of 50,000 to 100,000 gallons per acre per day, but the lower figure would be much nearer an average.

Intermittent Filtration of Clarified Sewage. The older intermittent filtration areas in Massachusetts, almost without exception, treat crude or only very roughly sedimented sewage; and crude sewage can be treated by this process with good results. The area required for a given community is, however, chiefly determined by the amount of sewage which can be passed during the winter months without scraping the beds; and this quantity is fixed by the amount of suspended matter in the sewage.

A dose of 100,000 gallons on an acre corresponds to a depth of less than four inches of sewage. With a bed of fairly coarse

sand, well-leveled and equipped with good distributors, a dose of well-clarified sewage disappears in half an hour and may be repeated as often as once every six hours without interfering at all with nitrification. The deposit must, however, be removed from such a bed four times as often as from one operated at the ordinary rate; and it is impossible to do this during freezing weather. Where unlimited areas of sand *in situ* are not available, it is worth while to subject the sewage to preliminary processes for the removal of solids and to operate the filters in a more intensive fashion. An outdoor intermittent filter at the Technology Experiment Station was operated with marked success at a rate of between 200,000 and 300,000 gallons per acre per day by applying carefully sedimented sewage in doses at intervals of about five hours (Winslow and Phelps, 1911).

One of the largest and best constructed plants in which the sewage is subjected to preliminary treatment for the removal of suspended solids is at Saratoga, N. Y. The intermittent filtration beds, twenty in number, — eighteen of them about one acre in area, the others somewhat smaller, — consist of sand with an effective size of 0.20 mm. and a uniformity coefficient of about 2. This sand extends to an unknown depth and the ground-water level is about 16 feet below the surface. There is only one line of under drains in each bed, at a depth of 6.5 feet, and a line of 10–15 inch drain, at a depth of 11 feet. In the main drain manholes were placed at the junction of the laterals, and at the end of all drains, which were turned up and carried above the surface of the bed, air vents were provided. In this way circulation of air can take place, it is believed, with beneficial effect in the reduced accumulation of carbonic acid gas in the body of the filters. On account of the depth of the sand, only about a quarter of the liquid discharged upon the beds finds an outlet through the drains, the greater part running down through the sand without appreciably raising the water table. In midsummer twelve beds are used daily, the gates being changed twice; during the remainder of the year eight beds are used each day, one shift of the gates being necessary.

The average daily amount of sewage per bed in use is about 140,000 United States gallons, applied in four doses. The entire field is kept in commission and the beds used alternately, so that the average rate per day for the field is about 60,000 gallons per

acre. Mr. Barbour, who constructed the plant, believes that double this rate could be maintained with equally good results.

In the central regions of the United States, sand of suitable quality for filter beds must often be transported from considerable distances. The intensive operation of the sand filter has therefore reached a high degree of development in these states, particularly in Ohio and in Wisconsin, the removal of solids from the sewage being generally accomplished by preliminary treatment in the septic tank. In Ohio, for example, there are now 26 intermittent filtration areas, 12 of which are dosed with sewage from which the solids have been more or less carefully removed.

Two good examples of the combination septic-intermittent-filter plant have been in operation in the town of Wauwatosa, Wis. The first of these, a small plant to serve the town itself, was built about 1901 by Alvord and Shields. It was visited by one of the authors (Winslow, 1905), during the winter of 1904–05, when some 200 connections had been made with the sewer system, yielding a flow of about 100,000 gallons per day. The septic tank, of concrete, sheltered by a brick roof, was approximately 15 feet by 50 feet by 10 feet deep, with a capacity of 40,000 gallons, giving a storage period of 10 hours. According to the town engineer, the tank was cleaned out twice a year, a quantity of combined scum and sludge equal to half its capacity being removed by dipping out with pails and by the use of a small rotary pump. The effluent appeared like a good septic sewage, dark gray in color and with no large particles.

The septic effluent passed to a dosing chamber in a separate small brick structure, to be discharged on sand beds by an automatic device. The beds were 6 in number, 30 feet by 60 feet, with a combined area of one-fourth acre, thus giving a rate of 400,000 gallons. The sand used was coarse and the results obtained were said to be excellent. In cold weather, however, it was the practice of the authorities to discharge the septic effluent directly into Menominee Creek without filtration.

In the same town was a larger plant of almost exactly similar construction (Shields, 1904), which showed what good results can be obtained by careful and efficient operation. The Wauwatosa County Institutions formed a group of five buildings, including two insane hospitals, an almshouse, a county hospital and a home for dependent children. The total population was about 3500,

and the water consumption, 400,000 gallons per day. A chemical precipitation system was put in in 1889, the dosing house and coagulating basin remaining as its monument. Then a septic tank was substituted; but it proved unsatisfactory, and in 1904 Mr. Shields built a new septic tank and filter beds. The new tank was 85 feet long, 20 feet wide and 8 feet deep, with a central longitudinal partition and 3 concrete baffles. The storage period, under the conditions existing in 1905, appeared to be 6 hours. With the addition of the old septic tank, 8 feet by 17 feet by 54 feet, it would be increased to 9 hours. Both tanks were of concrete, housed under low brick buildings with wooden roofs.

From the septic tank the sewage flowed through an inverted siphon to the filter beds, located on the farther side of a small stream. The beds were eight in number, arranged in two rows, with a controlling house in the center. The four corner beds were each 50 feet by 110 feet while the four center beds were 57.3 feet by 55 feet, being shortened to provide room for the distribution system. The latter was of the general pattern described on page 252, including a dosing chamber discharged by any one of eight 15-inch siphons, each connected with one bed. The rotation of the beds was controlled by an automatic device.

The filter beds were built up of 12 inches of coarse gravel, 12 inches of fine gravel and 12 inches of coarse sand, and were underdrained by four lines of 4-inch pipe. The carriers were of the usual type — two straight troughs in each bed with 3-inch square holes about 2 feet apart.

The plant, when seen in 1905, was carefully supervised by the superintendent of the institution, and was working in admirable shape. The siphons flushed perhaps once every 35 minutes in the morning, every 45 minutes in the afternoon, and once an hour at night, so that each bed was dosed once in from 4 to 8 hours. The total area was about 1 acre for the 400,000 gallons treated.

In spite of severe weather in January, the dose disappeared in twenty minutes after its application; but at intervals it was necessary to rest a bed for a few days by putting into the regulator a chute to shut out one of the dosing siphons. The effluent from the plant, as seen flowing into the Menominee Creek, ap-

peared clear and well purified (Winslow, 1905). No analyses were obtainable from either of these plants.

It must be clearly understood that the object of the preliminary treatment in such plants is solely the removal of suspended solids so as to diminish clogging. It has been claimed in England by Martin, Cameron and Fowler that the soluble elements in a septic effluent are more easily nitrified than those in fresh sewage. There is little evidence in favor of this view. On the contrary, there is clear evidence that the septic process may be carried so far that the effluent is less easily nitrified than it would be without such treatment. At Andover, Mass., for example, where the sewage is strong, and already twenty-four hours old when it reaches the disposal area, the introduction of a septic tank proved distinctly harmful. It seems probable that any trouble of this kind can easily be overcome by aeration between the septic tank and the sand filters. At Saratoga Springs, N. Y., the septic effluent is made to flow in a thin sheet over circular perforated sheet-iron plates sixteen feet in diameter, with the result that the oxygen saturation rises to 70 per cent. It is interesting to note, as indicating the avidity with which the organic matter in such a liquid absorbs oxygen, that the saturation value falls to 40 per cent by the time the effluent reaches the filters (Barbour, 1905).

Practical Results of Intermittent Filtration. The process of intermittent filtration has been extensively used in the northern part of the United States where natural deposits of glacial drift sand are available. In 1904, Fuller stated that 41 plants of this type were in operation, most of them in New England and about half in Massachusetts. Daniels (1914), ten years later, reports 36 intermittent filters in the state of New Jersey alone. Ohio as stated above has nearly as many.

In general the practical results obtained from these plants have been highly satisfactory. If a sand filter be overdosed, nitrification will be interfered with and clogging will ensue; but if the amount of sewage be properly adjusted to the sand in question the effluent will be clear (in favorable cases as bright as spring water), completely nitrified, and almost free from sewage bacteria.

The purification effected always varies somewhat with the season of the year, as pointed out above. The worst effluent is generally produced in the late winter and early spring when cold

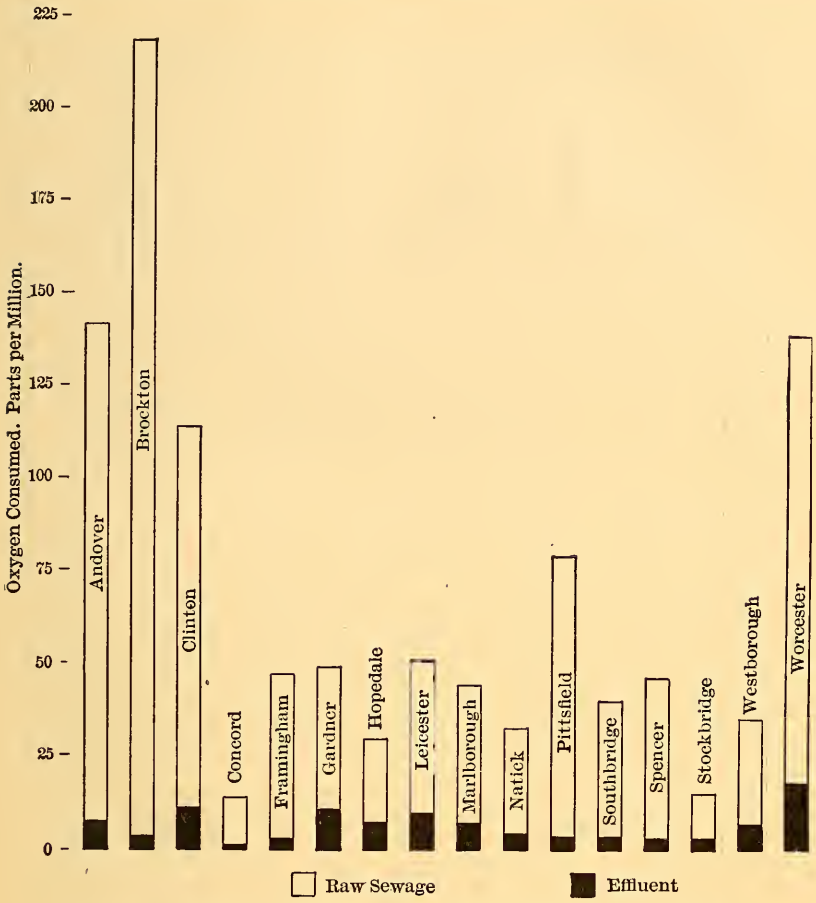

FIG. 75. Diagram of Purification Effected by Intermittent Filters.

has produced its maximum effect on the nitrifying organisms and when the surfaces of the beds are likely to be clogged by the solids accumulated during the winter. Since this is the time of year when stream flow is at its height an imperfectly purified effluent will be less objectionable than at other seasons.

TABLE LIV

STATISTICS OF INTERMITTENT FILTRATION IN MASSACHUSETTS

(Massachusetts, 1904; Worcester, 1905.)

Place.	Date of construction.	Average daily flow (million gallons).	Area of filter beds (acres).	Rate (million gallons per acre per day).	Population contributing sewage.	Preliminary treatment.	Material.	Average of monthly analyses, 1903 (parts per million).				
								Nitrogen as —				Oxygen consumed in 2 minutes, boiling.
								Ammonia N.	Albuminoid N.	Nitrates.	Nitrites.	
Andover	1898	0.125	3.65	0.0342	3,600	Screening; sedimentation.	Sewage	39.7	5.6	....	....	49
							Effluent	9.1	0.5	8.3	0.2	7.4
Brockton	1894	0.878	21.48	0.0409	25,000	Partial sedimentation	Sewage	43.8	12.9	....	....	219
							Effluent	1.9	0.1	30.8	0.1	3.3
Clinton	1899	0.785	23.5	0.0334	10,000	Partial sedimentation	Sewage	33.2	7.9	....	....	113.7
							Effluent	0.8	0.6	4.4	0.2	11.2
Concord	1898	0.312	3.3	0.0945	1,200	Screening	Sewage	5.7	1.4	....	....	13.6
							Effluent	0.01	0.09	8.5	0	1.1
Framingham	1889	0.652	19.9	0.0328	7,500	Screening	Sewage	26.1	6.5	....	....	47.3
							Effluent	1.8	0.2	9.9	0.2	2.6
Gardner (Gardner system).	1891	0.302	2.5	0.1208	3,500	Sedimentation	Sewage	20.2	4.9	....	....	49.2
							Effluent	14.8	0.7	0.4	0	10.7
Hopedale	1892	0.150	2.3	0.0652	2,000	Septic tank	Sewage	18.3	2.8	....	....	29.8
							Effluent	8.6	0.7	15.9	0.3	7.2
Leicester	1894	0.030	0.36	0.0833	500	Sedimentation	Sewage	22	4.1	....	....	50.8
							Effluent	5.8	0.7	9.1	0.4	9.6

TABLE LIV (*Continued*)

STATISTICS OF INTERMITTENT FILTRATION IN MASSACHUSETTS

(Massachusetts, 1904; Worcester, 1905.)

Place	Date of construction.	Average daily flow (million gallons).	Area of filter beds (acres).	Rate (million gallons per acre per day).	Population contributing sewage.	Preliminary treatment.	Material.	Average of monthly analyses, 1903 * (parts per million).				
								Nitrogen as —				Oxygen consumed in 2 minutes, boiling.
								Ammonia N.	Albuminoid N.	Nitrates.	Nitrites.	
Marlboro	1890	1.100	11.2	0.0982	10,000	Sedimentation	Sewage	25.9	4.4	....	....	44.4
							Effluent	10.2	0.5	3.5	0.3	7.3
Natick	1895	0.566	11.1	0.0510	4,000	Screening	Sewage	12.2	2.6	....	....	32.7
							Effluent	5.1	0.3	2.2	0.2	4.4
Pittsfield	1890	1.456	21.67	0.0672	15,000	Screening	Sewage	12	8.2	....	....	79.2
							Effluent	2.2	0.3	6.7	0.2	3.8
Southbridge	1899	0.350	7.25	0.0483	2,200	Screening	Sewage	16	3.5	....	....	40.1
							Effluent	3.2	0.3	2	0.1	3.6
Spencer	1897	0.375	9.3	0.0403	3,000	Screening	Sewage	1.5	4.5	....	....	46.5
							Effluent	1.2	0.2	3.7	0.2	3.3
Stockbridge	1899	0.075	3.6	0.0208	800	Screening	Sewage	9.8	1.6	....	....	15.2
							Effluent	0.9	0.2	1.4	0	3.3
Westboro	1891	0.282	4	0.0705	3,000	Screening, partial sedimentation.	Sewage	13.8	4.4	....	....	35.5
							Effluent	5.3	0.7	3.6	0.5	7.3
Worcester	1898	1.080	8.77	0.123	122,000	Sedimentation	Sewage	18	8.1	0.6	0.1	138.7
							Effluent	10.9	0.9	2	0.3	18.1

* Worcester, 1904.

A fair idea of the general practice of intermittent filtration in Massachusetts and of the amount of purification obtained may be gained from Table LIV, compiled from the Thirty-fifth Annual Report of the Massachusetts State Board of Health; and the data for "oxygen consumed" are plotted in Fig. 75 (Winslow and Phelps, 1906).

With regard to the comparative results of different Massachusetts plants the following points may be noted: The poor effluents at Westboro and Gardner were in part due to careless operation, the sewage being allowed to run on continuously for days. At Clinton the applied sewage was very strong. At Leicester, Andover and Hopedale the Board attributed results below the average to the fact that the sewage was stale or septic when applied. At Worcester the sewage was strong, and it is probable that acid-iron waste interfered with the process of nitrification.

TABLE LV

BACTERIA IN SEWAGE SEPTIC EFFLUENT AND SAND-FILTER EFFLUENT AT IOWA STATE COLLEGE

(Walker, 1901.)

Month.	Bacteria per c.c. Monthly averages.		
	Sewage.	Septic effluent.	Sand effluent.
August, 1899	2,392,600	1,388,300	2,246
September	8,815,000	3,245,000	3,660
October	6,064,800	4,941,000	4,320
November	4,537,333	3,014,000	2,261
December	816,333	848,000	2,319
January, 1900	848,000	726,000	830
February	345,533	233,810	3,451
March	132,125	112,500	2,480
April	2,121,000	1,392,800	13,263
May	1,021,000	783,300	3,077
June	1,318,100	1,391,300	2,359
July	3,908,700	4,578,333	2,270
August	403,118	215,700	546
September	1,181,533	383,733	850

The bacterial purification effected by an intermittent filter may be illustrated by Tables LV and LVI, the first showing the monthly averages obtained at Ames, Iowa, and the second the results of a series of tests, made by one of the writers on four different days during the autumn of 1908 at Brockton, Mass.

TABLE LVI

BACTERIA IN SEWAGE AND EFFLUENTS AT BROCKTON

Average of four examinations, autumn of 1908.

	Bacteria per c.c. gelatin, 20°	B. coli per c.c. lactose bile.
Sewage	3,150,000	150,000
Effluent A	1,900	400
" B	6,300	15
" D	125	0
" E	1,400	5
" F	2,000	1

The Cost of Intermittent Filtration. Intermittent filtration, like all other processes of sewage purification, cannot be carried out except at considerable expense. The first cost of beds built under most favorable conditions and where sand is used *in situ*, may be as low as $1000 or $1500 per acre. Where the sand cannot be used *in situ*, or in other words, where the beds are artificially constructed, the cost will be materially increased. $3000 per acre is perhaps a fair average cost under all conditions, but it not infrequently runs up to $5000 or even $10,000.

Intermittent filter beds are actually operated in Massachusetts at rates between 50,000 and 100,000 gallons per acre per day. By including a tank in which a large proportion of the suspended matter is removed, they may be operated at higher rates, as already stated.

Assuming $1250 as the cost per acre of filters built under the best conditions, and 75,000 gallons per acre per day as the rate of filtration, the annual interest charges, at $3\frac{1}{2}$ per cent upon the capital expenditure, will amount to $1.60 per million gallons. Assuming an average cost of $3000 per acre and the same rate of filtration, the annual interest charges, at $3\frac{1}{2}$ per cent, will amount to $3.84. While these rates indicate the interest charges which should be anticipated for such plants, built under the best and under average conditions respectively, these charges may, under less favorable conditions, run as high as $6.50, or even higher. At Brockton, the cost of the first twenty-three beds was $9234 for land and $50,301.97 for construction. The second set of seven beds cost $24,438.92 for construction; the third set of seven beds cost $23,239.06 for construction. The land for the

last two sets cost $10,510; but only a small part of this is in use. The total first cost of the whole plant, including the laboratory may be put at $122,665.95. Up to and including the year 1907, the capital cost was $99,426.89. The interest on this sum at 3½ per cent would amount to a capital charge of $3480; or $8 per million gallons of sewage treated in 1907.

The cost of operation of intermittent filtration plants is often considerably larger than the interest charges, mainly because the surface of the beds requires considerable care to maintain efficient purification. At Brockton, for example, the cost of labor at the disposal area in 1907 was $5745.22, of which $4355.03 was for the care of the surface of the beds. In addition, the laboratory and other minor items bring the total operating cost for that year to $6805.22. On the basis of 425,000,000 gallons of sewage treated this amounts to $16 per million gallons; or, plus interest charges on the capital cost of the plant, to $24 per million gallons.

These Brockton costs are undoubtedly high on a million gallon basis, on account of the strength of the sewage; but on a per capita basis, they correspond closely with Fuller's estimate (Fuller, 1909) of 20 cents per capita per annum. The expense of maintenance at 16 Massachusetts plants varied in 1903 from $0.61 per million gallons at Natick to $21.92 at Stockbridge (Massachusetts, 1904). The Natick beds received practically no care except in connection with cropping, but operated satisfactorily with a relatively small amount of sewage. The next lowest figures were $2.45 at Pittsfield and $2.60 at Marlboro (large plants) and $2.87 at Concord (weak sewage). The costs at Clinton, Southbridge, Spencer, Westboro and Worcester were between $3.91 and $7.80; at Gardner, over $9 at each of its two areas; at Leicester, $11; and at Andover, $13.98. It may be considered, perhaps, that $7.50 per million gallons is somewhere near a fair figure for the average American sewage, rising perhaps to $10 or $15 with a strong sewage like that of Brockton.*

A total cost of ten to twenty dollars per million gallons for operating expense and interest charges is not exorbitant when the excellent character of the effluent from intermittent filtration is taken into account. When a high degree of purification is

* All cost data in this and other chapters, are based on pre-war conditions; and it must therefore be remembered that their significance is only relative.

required, and when sand for filtration is available, this method unquestionably furnishes an ideal solution of the problem. If a less perfectly purified effluent will serve, other processes of treatment may prove more economical. The availability of intermittent filtration is, in any case, directly dependent upon the proximity of suitable sand areas. In regions where there are no deposits of good sand within easy reach the cost would generally prove prohibitive except for small communities.

CHAPTER X

TREATMENT OF SEWAGE IN CONTACT BEDS

Historical Development of the Contact Bed. The general method of sewage disposal worked out at Lawrence had only a limited application, being unduly expensive for regions not provided with adequate deposits of sandy soil. For large communities of this kind the cost of constructing sand filters of sufficient area to treat sewage, at a rate of 100,000 gallons per acre per day, would be prohibitive. In England, where sand is not of common occurrence, it was absolutely necessary to modify the process so as to obtain higher rates of filtration. It was in England, therefore, that the newer methods of sewage purification, the so-called "biological processes" were developed, based like intermittent filtration on the oxidizing activity of micro-organisms, but scientifically controlled and regulated so as to be more intensive in their action.

W. J. Dibdin, Chemist to the London County Council, was one of the first English sanitarians to grasp the essential principles of sewage purification. In studies of the self-purification of the Thames, H. C. Sorby had pointed out, as early as 1883, the part played by living organisms, although he had in view chiefly the consumption of solids by the larger microscopic forms. In 1884 Dupré went a step further in affirming the relation of organic life to the oxidations which take place in a purifying stream. Dibdin, who had been associated with both these observers, read a paper before the Institution of Civil Engineers in 1887, in which he worked out the whole theory as follows:

"In all probability the true way of purifying sewage, where suitable land is unavailable, will be first to separate the sludge, and then to turn into the effluent a charge of the proper organism, whatever that may be, specially cultivated for the purpose, and retain it for a sufficient period, during which time it should be fully aerated and finally discharged into the stream in a really purified condition. This, indeed, is only what is aimed at and imperfectly accomplished on a sewage farm."

The treatment of London sewage by chemical precipitation had been recognized by the Metropolitan Sewage Commission of 1884 as only a temporary expedient, purification by land filtration being contemplated as the ultimate outcome. As soon as the Massachusetts results were published, Dibdin saw that they gave promise of a better solution of the London problem than could be found in the laying out of immense sewage farms. The problem was to purify sewage on still smaller areas than those used in the Lawrence experiments. Obviously, in order to accomplish this end it was necessary to build filters of material coarser than sand or gravel. With coarse material, however, frictional resistance could no longer be depended on to delay the passage of sewage through the bed and give time for the purifying agencies to work. It was necessary, therefore, to regulate the flow by constructing water-tight filters, in which the sewage could somehow be retained in " contact " with the filling material and its accumulated growth of micro-organisms for the requisite period of time.

Dibdin's first experiments on the purification of sewage at high rates were carried out at one of the London sewage outfalls between May and August, 1892. Four wooden tanks were installed at the northern (Barking) outfall. Each was 5 feet deep and had an area of one two-hundredth of an acre. The tanks were filled, respectively, with burnt clay, pea ballast (Lowestoft shingle), coke breeze, and a combination of gravel and sand over a layer of proprietary material. All received effluent from the chemical precipitation tanks at an average rate of 400,000 gallons per acre per day. Sewage was allowed to run through continuously for eight hours, the rate being controlled by partially closing the outlet valves, and the beds were allowed to stand empty for aeration during the remainder of the twenty-four hours. All four filters yielded effluents which were purified to a very considerable extent, the oxygen consumed and albuminoid nitrogen values being reduced to a half or a third of the amount in the applied liquid. Of the four filling materials, the coke breeze proved most satisfactory, the coarser burnt clay yielding a much poorer effluent, and the sand clogging seriously and giving a clear but imperfectly purified filtrate.

Coke breeze was therefore fixed upon as the best material for further experiments, and a second series was begun, to study the

details of practical operation on a larger scale. A filter bed 1 acre in area, consisting of 3 feet of pan breeze covered with 3 inches of gravel, was constructed at Barking and put into operation in September, 1893. At first the bed was dosed too heavily and soon became clogged and foul. The need of rest and aeration, especially when a new filter is first operated, was thus clearly shown. After three months' rest the bed could handle two fillings a day, the sewage being allowed to stand in it for a period of from one to two hours. The cycle finally established allowed one and one-half hours for filling the bed, two hours for standing full, two and one-half hours for emptying, and six hours for aeration. When gradually worked up to its full capacity, sewage could be treated at a rate of 1,200,000 gallons per acre per day. Though the purification effected was not at all equal to that obtained by intermittent filtration, the results, as shown by the following table, were surprisingly good.

TABLE LVII

RESULTS OF CONTACT TREATMENT AT LONDON, ACRE FILTER

Parts per million. (Clowes and Houston, 1904.)

Period.	Oxygen consumed in 4 hours at 80° F.		Nitrogen as —			
			Ammonia N.		Nitrates.	
	Chemical effluent.	Filter effluent.	Chemical effluent.	Filter effluent.	Chemical effluent.	Filter effluent.
September–December, 1893.	59	17	4.8	1.4	1.6	1.9
April–June, 1894	59	12	4.9	1.1	1.8	3.4
July–November, 1894	52	10	4.7	1.3	0.3	2
January–March, 1895	61	14	4.8	1.4	5.4	9.7
April–September, 1895	46	9	4	1.3	2	7.6
May–June, 1897	31	6	3	0.9	0.6	4.1
1900–1901	55	9	....	....	0.4	10

The early London experiments of Dibdin were greatly extended by Clowes and Houston. Various details of construction and operation were worked out at both the Barking and Crossness outfalls, and the recommendation was finally made that the present plant for chemical treatment be abandoned and that the London sewage be, first, settled to remove gross mineral matter; second, septicized for six hours; and, third, treated in single-contact beds of coke, 12 feet deep, at a rate of 5,200,000

gallons per acre, attained by four fillings per day (Clowes and Houston, 1904).

In 1894, as a result of the first Barking experiments, Dibdin installed seven experimental contact beds at Sutton, in Surrey. Here, two important modifications of the contact system were introduced. In the first place, the sewage was subjected to successive treatments, first in coarse and then in fine-grained beds, now known as the "double contact" system. In the second place, after the process had worked well with chemical effluent, as it had done at London, the treatment of crude sewage was attempted. Beginning November, 1896, a double-contact system treating crude sewage was operated for the first time. The depth of the beds was 3 feet 6 inches, and the filling material, burnt ballast, larger than three-eighths inch. Two fillings a day were made, giving a rate on each individual bed of 900,000 gallons per acre. The analytical results showed a reduction of oxygen consumed from 76 parts per million in the sewage to 26 parts in the effluent of the first bed and 10 parts in the effluent of the second bed (Dibdin, 1903).

The Physics and Chemistry of the Contact Bed. Dibdin's original idea was merely to accelerate the process of intermittent filtration so that small beds of coarse material might take the place of larger sand areas; and it was at first assumed that the fundamental processes involved were essentially the same.

Dunbar and Thumm (1902), in experiments at Hamburg, have since shown that the reactions in the contact filter, as a result of the alternate aerobic and anaerobic conditions, follow a peculiar and characteristic course, and that while the changes that go on in the empty period resemble those of the intermittent filter, they are complicated by a widely different set of reactions in the full period. Both physical and chemical changes play an important part. While the bed stands full, the solids in the sewage collect on the surface of the filling material and the soluble constituents are to a large extent adsorbed by the bacterial jelly with which the material is coated, or changed to insoluble colloids. The adsorption takes place in virtue of the general tendency exhibited by colloidal films to remove substances from contiguous solutions. Dunbar offered striking evidence of the importance of this physical factor by a series of experiments in which he determined the minimum time neces-

sary to produce purification. With a well-ripened filter he found that the oxygen-consumed value was reduced 83 per cent by five minutes' contact (Dunbar, 1908). This was confirmed by Frankland, who found that a value for oxygen consumed of 555 parts per million for raw sewage was reduced to 93 in five minutes. It was still 93 after thirty minutes and 49 after twelve hours.

The retention of the organic matter by a contact bed is purely physical; and some investigators (Bredtschneider, 1905) have even maintained that the whole action of the contact bed is a mechanical one. On this view the slow ripening of a filter, which clearly points to bacterial growth, would depend merely on the necessity for the formation of zoögleal films in which the adsorptive process could readily take place. It is quite clear, however, that the adsorbed material must afterward be chemically changed if any real purification of the sewage as a whole is to be effected. That this change is due to bacterial action is indicated by the fact that chloroform and mercuric chloride quickly put a stop to it (Dunbar, 1908). The chemical processes set up are much more complex than in the intermittent filter; for it has been shown by Dunbar and Thumm (1902), and Phelps and Farrell (1905), that there are at least three distinct reactions involved — nitrification during the empty period and hydrolytic splitting and denitrification during the full period.

In the nitrifying period the bacteria set in train processes essentially similar to those of the intermittent filter, which may be indicated by the following generalized formula:

Reaction 1. Nitrification:

(*a*) $2\,R\cdot N:R' + 3\,O_2 = R\cdot O\cdot R\cdot + 2R':O + N_2O_3$;

(*b*) $N_2O_3 + O_2 = N_2O_5$.

At the end of this period considerable quantities of nitrates are present in the filling material of the contact bed, and the amount of nitrates increases with the length of the period. When the bed is refilled the same action continues for a time. Soon, however, the supply of oxygen is consumed, active nitrification stops, and anaerobic putrefactions begin, causing hydrolytic splittings of the following type:

Reaction 2. Hydrolysis:

$$R\cdot N:R' + 2\,H_2O = R\cdot OH + HO\cdot R'\cdot NH_2.$$

At this stage the contact filter has the liquefying properties of the septic tank. There is a bacterial reduction of the nitrates to nitrites, and a formation of partly reduced nitrogenous bodies, primary amines, etc. This leads to a decomposition of the nitrites present and the liberation of gaseous nitrogen according to the following formula:

Reaction 3. Denitrification:

$$2\,HO \cdot R' \cdot NH_2 + N_2O_3 = 2\,R'(OH)_2 + 2\,N_2 + H_2O.$$

In intermittent filtration the nitrogenous constituents of sewage may be almost quantitatively converted into nitrites and nitrates. In the contact bed, however, it is obvious that the nitrates found in the final effluent give no true measure of the purification effected, since under ideal conditions the nitrates formed from half the nitrogen during the empty period, according to reaction 1, above, would be exactly used up in decomposing the other half during the full period, according to Reaction 3. Dunbar and Thumm (1902) found as a matter of practical experience that the highest purification frequently accompanied the lowest nitrate content in the effluent. A part only of the nitrogen appears as nitrates, a part remains in a stable organic state; and a considerable part is lost in gaseous form. Clark (1903) has pointed out that the loss of nitrogen amounts to from 38 to 50 per cent, and Phelps and Farrell (1905) found a loss of from 35 to 50 per cent.

From the chemical standpoint the decomposition of organic matter into free nitrogen, which takes place in this type of filter, is admirable in the completeness of the effect produced with a minimum consumption of oxygen. The alternation of two or more different processes in the same culture chamber is not, however, apt to lead to the best results. Constant conditions are generally essential to the most successful bacterial action; and in such a mechanism as a contact bed, which depends on a delicate adjustment between several groups of micro-organisms, there must often be serious impairments of efficiency due to an unbalancing of their proper equilibrium. It is therefore particularly unfortunate that practically nothing is known of the actual bacterial types concerned in the functioning of the contact bed.

Construction of Contact Beds. A contact bed is a water-tight basin, generally built of cement concrete. In England the at-

tempt has been made to utilize simple excavations in clayey soil for this purpose; but leakage, settling and the working up of clay into the bed have generally introduced serious complications.

In making contact beds the ground is excavated to the depth of about four feet and the sides and bottom made water-tight — best by cement concrete six inches thick. The bottoms of the beds are suitably graded and an ample number of underdrains provided to ensure quick draining. The effluent from these drains passes into a main drain, which is so constructed that by the use of manholes and valves the effluent can be carried from a high-level to a low-level bed, or can be delivered directly into the effluent channel. The construction is such that the effluent channels can be kept full of the purified sewage or completely emptied. The sewage carriers are so arranged that the sewage can be delivered upon any of the beds, either high or low level.

The size of a single bed should be so proportioned to the dry-weather flow of sewage that it will not contain, at most, more than two hours' flow unless the sewage is applied from dosing tanks; for if it requires a long time to fill the beds the sewage that last enters is apt to be drawn from the bed before sufficient time has elapsed to produce the best results. For large plants, one-half acre has been fixed on in England as the maximum size. In the United States the contact beds at Plainfield, N. J., and Mansfield, Ohio, are between one-fourth and one-fifth of an acre in area. At Canton, Ohio, the beds are one-half acre each and at Alliance, Ohio, one-quarter acre.

The total volume of the beds to be provided for a single-contact system should be from one to one and one-half times the daily sewage flow, as the liquid capacity of a bed in operation will equal about one-third of its volume, and a bed can, as a rule, only receive two to three doses of sewage per day.

The usual depth allowed for contact beds is from three to four feet. The Royal Sewage Commission advises that they should not be deeper than 6 feet nor shallower than 2 feet, 6 inches. Clowes and Houston (1904) reported from their London experiments that beds 3 feet, 5 feet and 13 feet in depth gave equally good effluents. Most other observers have found, however, that the better aeration obtained in shallow beds was distinctly beneficial. Studies at Exeter, in which samples were

taken from taps placed at different depths in a contact filter, showed the best results at 3 feet below the surface, and at Manchester a 15-inch bed gave specially good results. In Ohio the plants at Canton, Alliance and Mansfield all have 5-foot depths of filtering material.

The underdraining of the contact bed must be so laid and of such size that the bed can be completely emptied in one-half hour or less. To accomplish this, contact beds have at some

Fig. 76. Underdraining Contact Beds at Manchester.

places been constructed with what are practically false bottoms made of tiles, separated from each other by a space of an eighth to a quarter of an inch. At other places the drains are laid in the cement concrete and covered with perforated tiles so as to prevent the falling of the filling material into the drain. Sometimes the drains are of pipe with open joints, laid sufficiently close together to ensure quick drainage.

Filling Material for Contact Beds. Almost any hard, nonfriable material of the proper dimensions may be used for filling material in contact beds. Among the substances used in England

are burnt clay, coal, coke, gravel, broken bricks, clinker, granite, sandstone, saggers (from pottery works) and furnace slag. Certain of the earlier experiments, like those of Clowes and Houston at Barking and those of the Massachusetts State Board of Health at Lawrence, indicated that coke was a particularly favorable material. Coal proved best in other experiments. At Birmingham, for example, it was found that the purification, measured by reduction in oxygen consumed, was 64 per cent with broken stone, 71 per cent with slag and 93 per cent with coal (Bredtschneider and Thumm, 1904). Where coke was used with success, it was held to be in virtue of its rough and porous surface, and Clark and Gage have reported particularly poor results with very smooth materials. Other investigators emphasize the chemical rather than the physical nature of the filling material used. Thus at Manchester (1901) and Hamburg (Dunbar, 1908) the presence of iron in the filling material proved beneficial. An excess of iron however was objectionable. Cleaned, screened slag was used for 8 acres of contact bed filling at Canton, Ohio, and this material was also used to replace in part, the cinders at the Alliance, Ohio, plant, which had badly disintegrated following four years' use. A careful test of the quality of the slag was made before its use was authorized, in order to guard against this material cementing together or disintegrating. On the whole it is probable that the choice of filling material will be best controlled by local conditions of cost and convenience, although material that is liable to disintegration should be avoided if possible.

The size of the filling material used in a contact bed is of prime importance. Both adsorptive efficiency and biochemical activity depend directly on the aggregate of the exposed surfaces of the fragments in the bed; and the smaller the fragments the greater will be the surface. Hering (1909) calculates that at Wilmersdorf a coke bed of 5-inch material has 25 square feet of bacterial surface to every cubic foot; at Birmingham, $1\frac{3}{4}$-inch slag and granite give 60 square feet of surface per cubic foot of bed; and at Hanley, $\frac{3}{8}$-inch saggers give 135 square feet per cubic foot. Aside from this factor of bacterial surface, the bed of finer material exerts a considerably greater mechanical straining action and thus removes a larger proportion of suspended solids.

The following table from Dunbar gives a good idea of the relation between size of material and purification effected:

TABLE LVIII

EFFECT OF CONTACT TREATMENT WITH MATERIAL OF DIFFERENT SIZES

(Dunbar, 1908.)

Size of material.	Per cent purification, oxygen consumed.	
	Coke.	Gravel.
2– 3 mm.	70.2	61.8
3– 5 mm.	69	61.8
5– 7 mm.	64.6	57
7–10 mm.	62.5	56.6
10–20 mm.	51	46.5

In general it may be said that material of half-inch size or less will sometimes yield a stable effluent, while larger material will rarely do so. On the other hand, fine-grain beds clog very rapidly, and it is necessary to take out the material and wash it at more frequent intervals. The most economical balance between these two considerations has never been scientifically worked out, and different engineers have widely varying opinions. Clowes and Houston (1904), as a result of their London experiments, recommended the use of "walnut-size coke." In evidence before the Royal Sewage Commission, Fowler recommended one-eighth inch material, Cameron one-eighth to one-half inch, Frankland one-eighth to three-fourths inch, and Dibdin one-half to four-inch for first contact and one-sixteenth to three-eighths inch for second contact. Barwise (1904) suggests the use of coarser filling — three- to five-inch material for primary beds to treat septic effluent and one-half to one and one-half for secondary beds.

The amount of suspended matter in the liquid treated is of course one of the controlling features in deciding this point. The Royal Commission on Sewage Disposal (1908) summarizes its conclusions as follows:

"With a crude sewage containing 40 parts per 100,000 of suspended matter, the material will probably have to be from three inches upwards in diameter, and even then sludge will accumulate on the top.

" With a septic-tank liquor containing 8 to 10 parts per 100,000 of suspended matter, material of a diameter from three-eighths to five-eighths of an inch may probably be used effectively; while with a good precipitation liquor containing from 1 to 3 parts of suspended matter, the best results will probably be obtained from material as fine as one-fourth inch diameter.

" It is, however, impossible to make any but the most general statement as to the most suitable size of material for contact beds, as, in some cases, there may be special circumstances which affect the question, such as the character of the suspended matters or the smoothness of the filtering material."

In general the most successful arrangement for a contact plant will include a double-contact system with rather coarse material, over an inch in diameter, for the primary bed and with finer ma-

TABLE LIX

DATA IN REGARD TO THE CONSTRUCTION OF ENGLISH CONTACT BEDS

(R. S. C., 1908.)

	Suspended matter in the liquor treated (approximate figures).	Nature of material.	Size of the material in the contact beds.
	Parts per million.		Diameter.
Crude sewage			
Hampton	485	Clinker	Above 4″
Leeds	350	Clinker and coke	Above 3″
Newton-le-Willows	300	Clinker	Top 18″–$\frac{1}{2}$″ to $\frac{1}{4}$″ Body of filter —2″ to 1$\frac{1}{2}$″
Withnell	200	Clinker	1″ to 1$\frac{1}{2}$″
Maidstone	140	Clinker	Above $\frac{3}{4}$″
Settled sewage			
Oswestry	About 200	House coke	1$\frac{1}{2}$″ to $\frac{1}{2}$″
Halton	110	Clinker and pebbles	1″ to 2″
Septic tank liquor			
Leeds	180	Clinker	$\frac{5}{8}$″ to $\frac{3}{8}$″
Guildford	160	Burnt ballast	$\frac{1}{2}$″ to 3″
Hartley Wintney	150	Clinker	$\frac{1}{2}$″ to $\frac{1}{8}$″
Exeter (Main)	About 140	Clinker	$\frac{1}{2}$″ to 1″
Andover	120	Clinker	$\frac{1}{8}$″ to $\frac{1}{2}$″
Exeter (St. Leonards)	85	Clinker and coke	$\frac{1}{2}$″
Slaithwaite	80	Clinker	Top foot —$\frac{3}{8}$″ to $\frac{1}{3}$″ Body of filter —$\frac{3}{8}$″ to 1″
Precipitation liquor			
Calverley	120 to 140	Clinker	$\frac{1}{4}$″ to $\frac{1}{2}$″
Kingston (experimental beds)	20	Clinker and coke	Coke bed, $\frac{1}{4}$″ to 1″ Clinker bed, $\frac{1}{4}$″ to $\frac{3}{8}$″

terial for the secondary beds. The solid matter of the sewage accumulates mainly in the first bed, the coarseness of which insures a reasonably long life, while the secondary fine bed provides for good purification.

Since the accumulations in the coarse material, after being allowed to dry, can be flushed out, the secondary beds, if used, should not be filled with material too fine to permit this flushing out action to take place if desired.

Operation of Contact Beds. With regard to the operation of contact beds, the number of fillings is the first point to be considered. At Hamburg it was found that for single contact two fillings a day gave the best results, while for double contact six fillings of the primary beds and three fillings of the secondary beds were recommended (Dunbar and Thumm, 1902). In the Barking experiments it appeared that two fillings a day gave better results than one; apparently a single filling does not maintain the bacteria at their maximum effectiveness. Birmingham experiments have indicated three fillings a day as effective, to be cut down to two if specially high purification is desired (Watson, 1903). At Crossness it was found that London sewage could be purified with as many as four fillings.

The distribution of fillings at regular intervals over the twenty-four hours does not appear to be a necessity. At Manchester contact beds were operated for two months with four six-hour cycles, and then for three months with four cycles in ten hours, followed by fourteen hours' rest. The results, as shown in the table below, were better by the second method.

TABLE LX

RESULTS OF OPERATION OF CONTACT BEDS AT MANCHESTER, ENGLAND

(R. S. C., 1902.)

Mode of operation.	Analyses of effluent (parts per million).			
	Oxygen consumed in 4 hours at 80° F.	Nitrogen as —		
		Ammonia nitrogen.	Albuminoid nitrogen.	Nitrates.
4 cycles in 24 hours	29	16.8	1.5	2.6
4 cycles in 10 hours	22.3	14.8	1.1	6.3

The next point of importance is the relative amount of time to be alloted to the four stages of the process — filling, standing-full, draining and standing-empty. Dibdin adopted two hours for the full-period and this is perhaps the general English practice. In Germany, too, Schumburg and others advocated this period (Bruch, 1899). Harding at Leeds found that one hour gave inferior results, while four hours was no better than two (R. S. C., 1902). Roscoe and Cameron, on the other hand, suggested shortening the period to one hour (R. S. C., 1902). Perhaps the commonest cycle adopted in the past is one which occupies eight hours, and allows one hour for filling, two hours for standing-full, one hour for emptying and four hours for aeration. Recent experience has suggested the wisdom of decreasing the full-period even more than suggested by Roscoe and Cameron, perhaps even to a half-hour. The filling and draining periods are of little importance in purification and should be shortened as much as possible (so long as the material is not disturbed by too violent a flow). The empty period is the one in which the major part of the purification takes place. The larger the proportion of the cycle which it occupies the better.

Regularity in the operation of the contact bed is obviously essential and this can often be most conveniently secured by the installation of automatic dosing devices. The filling and emptying of the beds, at definite intervals and in regular rotation, must be provided for in any device of this sort. The inflow to the beds is generally controlled by a series of float chambers, so connected by levers or air-lock siphons that when one bed is full the flow is diverted to the adjoining one. Sometimes the outflow is regulated by the same floats, so that when one bed is full and the one next in order begins to receive its dose the bed last treated begins at the same time to empty. One of the earliest of these devices, designed by Cameron for use at Exeter, is shown in Fig. 77. This gear controls four contact beds. The inlet and outlet valves of beds 1 and 2 are governed by a single lever, which is actuated by the floats in beds 3 and 4 and *vice versa*. Take the cycle at the time when bed 1 is full, 3 filling, 2 empty and 4 emptying. When bed 3 becomes full, the inlet of 2 and the outlet of 1 are opened. The sinking of the float in bed 1 then closes the inlet of 3 and the outlet of 4 by the lever on the opposite side of the gear. Thus bed 3 stands full and bed

4 empty, 2 fills and 1 empties, during the next period. Proper adjustment of piping with a regular flow of sewage will give approximately a two-hour period for each process in the cycle.

This system has the defect that filling and emptying periods must be made as long as the periods of standing full and empty, which is undesirable. The whole cycle will vary in length with

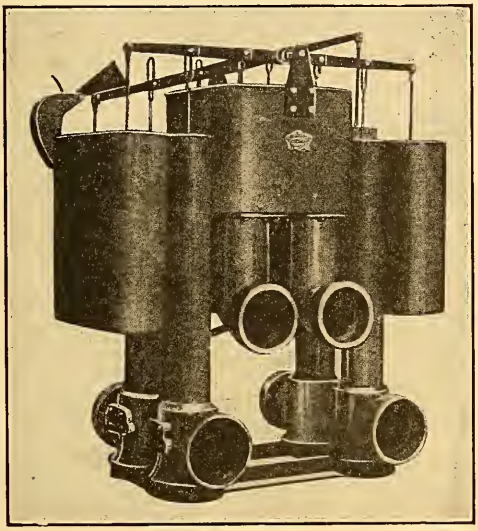

FIG. 77. Cameron's Alternating Gear for Dosing Contact Beds (courtesy of Cameron Septic Tank Company).

the rate of sewage flow. In many plants the arrangement is therefore modified, so that while the rotation of the beds is controlled by the inflow of sewage the effluent gates are governed by time siphons designed to discharge after a definite period of contact. This is generally effected by filling the effluent siphon chamber from the bed by a small pipe which takes a definite time to deliver the contents of the chamber. Thus the filling and emptying of each bed may be made as rapid as desired and the standing full period may be fixed at two hours, any variations in the rate of flow being taken up by the more elastic and longer period of aeration.

In England the trend of opinion is generally adverse to automatic operation of sewage plants. The Royal Commission in its final report (1908) concludes: "Our own observations and the

experience of others show that it is not possible to rely entirely on automatic apparatus for sewage works, although within certain limits it may be advantageously used with considerable saving of labor. In the case of large sewage disposal works, where men are always available, we consider that it would usually be inexpedient to provide an automatic plant." Important improvements have been made in the design of automatic devices in recent years; and in the United States there is a growing tendency to rely upon such appliances even for large installations.

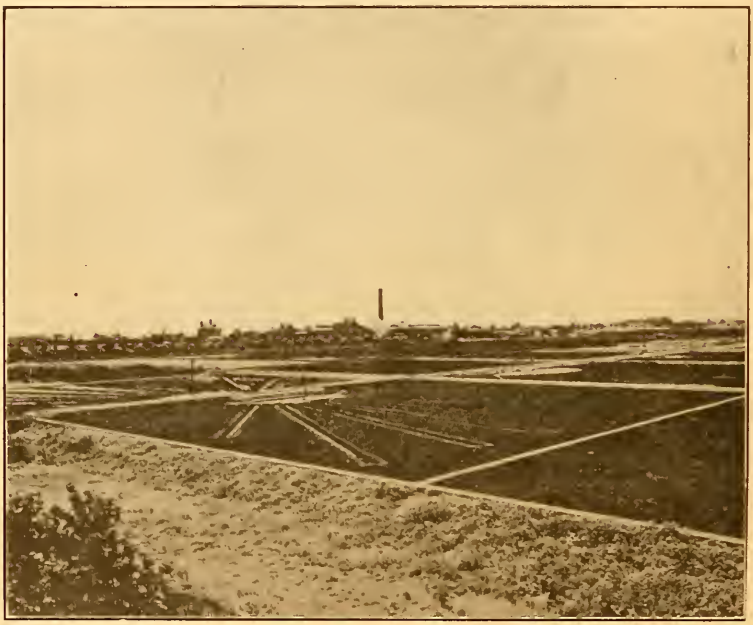

FIG. 78. Distribution of Sewage on Contact Beds at Manchester.

For small plants, where the cost of manual operation would be proportionately high, some measure of automatic control is almost an essential; although the necessity for occasional expert supervision should in no case be forgotten.

The distribution of sewage in small contact beds requires no special provision. In large installations surface troughs are often provided to avoid lateral filling. At Manchester the sewage is discharged in radiating surface channels lined with fine screen-

ings, which retain the bulk of the suspended matter and thus prolong the life of the beds themselves (Fig. 78).

At Plainfield, N. J., and at Ballston Spa, N. Y., the sewage was at first delivered through one-half foot pipes laid about one foot below the surface (Fig. 79). Using this method the surface of the bed remains clean, and there is less danger of a nuisance being created from odors, or of pathogenic bacteria being carried away from the beds by winged insects. Fuller (1912) carries this same principle further by filling contact beds from below

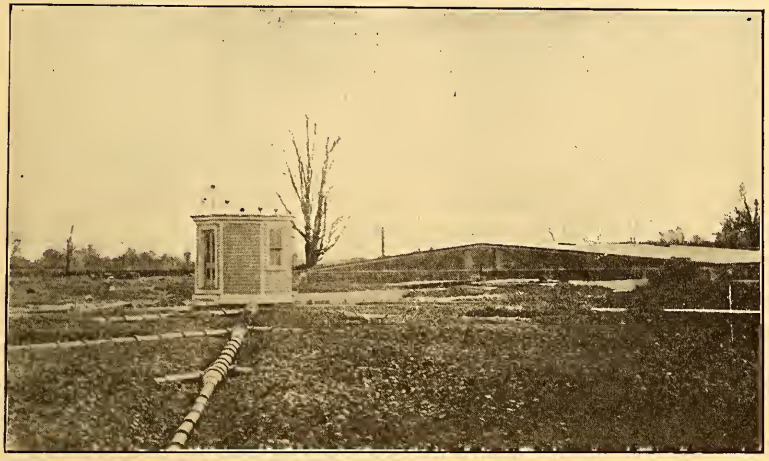

FIG. 79. Distribution of Sewage on Contact Beds at Plainfield, N. J. (courtesy of F. E. Daniels).

through a false bottom, never permitting the liquid to rise to within 6 inches of the surface. The primary beds at Plainfield have been filled in this way since 1910.

Results of Contact Treatment. In appearance the contact effluent is fairly clear, but usually rather dark in color. In regard to chemical composition, the purification effected by single and double contact treatment is fairly represented by the data collected from various sources and tabulated and plotted in diagrammatic form by Winslow and Phelps (1906) (Fig. 80). The first contact removes somewhat more than half of the organic constituents of the sewage, as measured by oxygen consumed and albuminoid nitrogen, and two-thirds or more of the suspended solids, while the second contact effects almost as great

TABLE LXI

RESULTS OF SINGLE AND DOUBLE CONTACT TREATMENT

Parts per million. (Winslow and Phelps, 1906.)

	Solids.						Nitrogen as —												Oxygen consumed in 4 hours at 80° F.*		
	Total			Suspended.			Ammonia N.			Albuminoid N.			Nitrites.			Nitrates.					
	Sewage.	First contact.	Second contact.	Sewage.	First contact.	Second contact.	Sewage.	First contact.	Second contact.	Sewage.	First contact.	Second contact.	Sewage.	First contact.	Second contact.	Sewage.	First contact.	Second contact.	Sewage.	First contact.	Second contact.
Lawrence: **																					
Nos. 137–163	....	....	....	....	....	....	36.3	24.3	12	5.9	2.4	1.4	....	....	....	....	8.3	14.6	42.6	21.2	10.3
Nos. 137–164	....	....	....	....	....	....	36.3	24.2	8.6	5.9	2.4	1.2	....	....	....	....	8.3	21.7	42.6	20.4	9.4
Aylesbury †	1190	1020	920	367	112	0	56.4	42.7	23.6	6.5	4	1.9	....	0.3	0.9	....	0.6	13.3	57.5	35.1	18.4
Blackburn ‡	1080	602	596	474	60	3	23.3	17.3	11.4	2.8	1.7	1	....	0.1	0.2	....	0.4	1.1	35.8	23	12.6
Leeds: §																					
Nos. 1 and 2	1670	1080	980	584	165	27	20.2	12.9	3.7	9.5	3.7	0.9	....	....	....	....	....	5.6	115	36	7
Nos. 3 and 4	1690	1180	1050	614	180	47	23.8	10.8	6.8	11.3	3.4	1.3	....	....	....	....	....	3.7	126	30	9
Nos. 5 and 6	1780	1170	1020	661	196	47	24.1	14.5	7.8	11.8	4.7	1.7	....	....	....	....	....	1.8	132	42	11
Leicester §§	1320	1070	1020	296	73	23	....	....	....	10.6	4.2	2.5	....	....	....	0.6	0.9	4.6	83.9	33.4	10.7
Manchester: ‖																					
Beds A and D	....	....	....	....	....	....	24	9.1	3.7	3.7	1.4	0.6	....	0.4	0.2	....	5.6	10.6	83.9	23.1	7.5
Beds C and D	....	....	....	....	....	....	24	8.5	3.6	3.7	1.3	0.6	....	0.3	0.2	....	6.3	10.8	83	19.6	7.5
Sutton ‖‖	1630	895	1000	824	45	6	94.2	32.6	8.3	7.3	2.9	1.4	....	1.5	0.7	....	4.1	19.7	56.9	21.9	9.2

See Notes on page 287.

NOTES TO TABLE LXI

* At Lawrence: 2 minutes' boiling.

** Lawrence: No. 137–163, run for 18 months during 1901–1902; area of beds, $\frac{1}{2000}$ acre each; first contact, broken stone $\frac{1}{2}$ to 1 inch, taking raw sewage, rate 0.9; secondary bed, fine coke, rate 0.7 (Fuller, 1905). No. 137–164, run for 6 months during 1901, like No. 137–163, except that coke was considerably " finer " in secondary bed (Fuller, 1905).

† Aylesbury: Experiments during first 6 months of 1898, weekly analyses; no further data given (R. S. C., 1902).

‡ Blackburn: No further data (R. S. C., 1902).

§ Leeds: Nos. 1 and 2, experiments during October, 1898–October, 1899; beds about $\frac{1}{8}$ acre in area; primary contains 5 feet of coke over 3 inches in diameter, receives raw or settled sewage at rate of 0.8; secondary, 6 feet coke, $\frac{1}{16}$ to $1\frac{1}{2}$ inches, rate like primary. Nos. 3 and 4, experiments during November, 1898, to June, 1900; beds each about $\frac{1}{8}$ acre in area and 3 feet deep; primary bed filled with clinker $\frac{1}{2}$ to 1 inch, and secondary $\frac{1}{16}$ to $\frac{1}{2}$ inch; rate during first 6 months, 1.1 (settled sewage), and during remainder of period 0.3 (raw sewage). Nos. 5 and 6, experiments during March to November, 1899; beds each $\frac{1}{8}$ acre in area; primary bed, 3 feet of clinker, 1 to 2 inches; secondary bed, 3 feet of clinker, $\frac{1}{16}$ to $\frac{1}{2}$ inch; rate, 0.8, raw sewage on each bed; 4 hours' contact during a portion of the time (Leeds, 1900).

§§ Leicester: Process No. 14; experiments during November, 1898, to July, 1899; primary beds, $\frac{1}{10}$ acre in area, filled with clinker $1\frac{1}{4}$ to $2\frac{1}{4}$ inches, $4\frac{1}{2}$ feet deep; secondary bed, $\frac{1}{65}$ acre in area, filled with 3 feet of clinker, $\frac{1}{8}$ to $\frac{1}{2}$ inch; rate, 1.6 on each bed, septic sewage (Leicester, 1900).

‖ Manchester: Experiments during January 4, 1900, to March 27, 1901; A and C were primary beds, the combined effluents of which were run on to D, area of each bed, $\frac{1}{77}$ acre on the surface; slope inward, 2 : 1. A contains 3 feet of clinker $\frac{1}{4}$ to $\frac{3}{4}$ inch; septic sewage at rate of 0.3. C contains 3 feet of clinker $\frac{1}{4}$ to $\frac{3}{4}$ inch; septic sewage at rate of 0.4. D contains 3 feet of clinker $\frac{1}{8}$ to $\frac{1}{2}$ inch; rate, 0.7 (Manchester, 1901).

‖‖ Sutton: Experiments from October, 1897, to August, 1898, coarse bed 3 feet deep, of burnt ballast; 5 beds of burnt clay; rate in each bed, 1 (R. S. C., 1902).

Rates expressed in million gallons per acre per day.

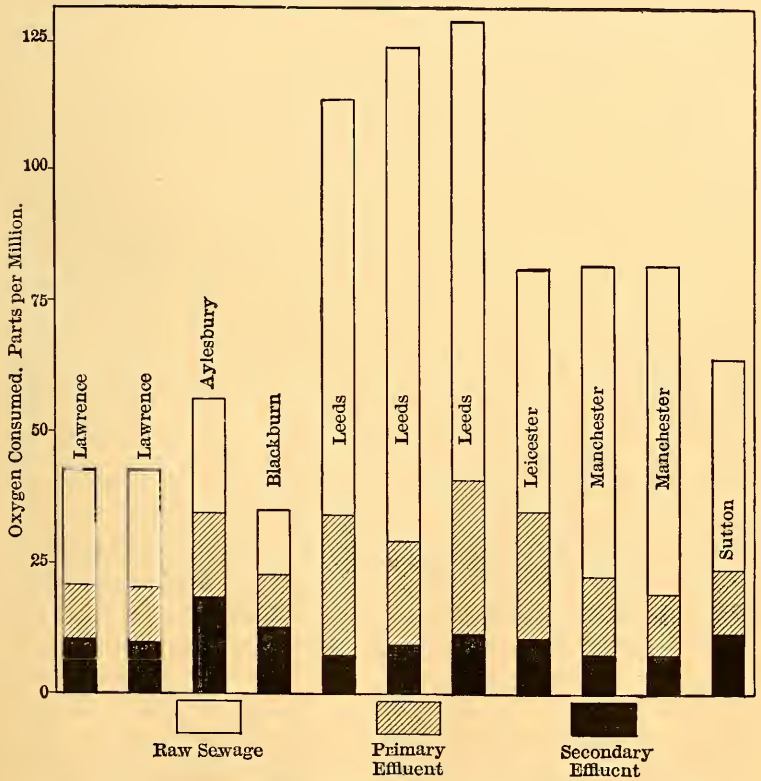

FIG. 80. Purification Effected by Single and Double Contact Beds.

a purification on the first-contact effluent. Aylesbury and Blackburn show the worst results among the English plants as far as ratio of purification is concerned. It will be noticed that these are the weakest sewages, and in all sewage treatment the last fractions of organic matter are the most difficult to remove. Except at Lawrence the nitrate content of the effluent is rather low, notably at Leeds and Leicester.

The effluent of the first-contact process, as is obvious from the analyses in Table LXI, generally contains too much organic matter to be considered as satisfactorily purified, while two successive treatments produce, as a rule, an effluent which is non-putrescible. Double contact is also more effective than a single treatment at half the rate, as indicated by the Manchester results tabulated below; and a double-contact system, with coarse primary and fine secondary beds, is probably the best form of this general method of sewage treatment.

TABLE LXII

RESULTS OF DOUBLE AND SINGLE CONTACT TREATMENT AT MANCHESTER, ENGLAND

Parts per million. (Manchester, 1904.)

	Nitrogen as —			Oxygen consumed in 4 hours at 80° F.
	Ammonia N.	Albuminoid N.	Nitrates and nitrites.	
Septic effluent	25.8	2.5	.....	70
First contact	14.5	1.2	0.5	22
Second contact	4.1	0.5	8.5	7
Septic effluent	31	3.5	.....	80
Single contact (one-half rate)	13.3	1.3	4.3	16

In some cases an attempt has been made to improve the effluent still further by adding a third contact treatment, as, for instance, at Hampton Court. What may be expected from this plan is indicated in the table on page 289. In general, the improvement in successive treatments progressively lessens, so that the results obtained are scarcely commensurate with the cost. The head required for successive contacts also introduces a serious factor.

With weak sewages, or a large dilution factor, single contact treatment may produce an effluent satisfactory for all practical

purposes; and this possibility is worthy of consideration where the head available is insufficient for the installation of a double-contact system.

Rate of Treatment in Contact Beds. The rate of treatment in contact beds is of course a function of the depth and the number of fillings (aside from the effect of the size of filling material and the composition of the sewage). It would be reasonable to measure contact rates in such units as acre-yards, which take depth into account. On such a basis Fuller (1912) estimates that a non-putrescible effluent can be obtained at a daily rate of 125,000–150,000 gallons per acre-foot. On a cubic basis this would mean that each cubic yard of filter volume could handle about one-half its volume of sewage per day.

For uniformity with sand filter computation, it is more common to calculate the rate of contact treatment on the basis of superficial area, assuming a reasonable depth. With a bed 3

TABLE LXIII

RESULTS OF TRIPLE-CONTACT TREATMENT

(Parts per million.)

	Solids.		Nitrogen as —			Oxygen consumed in 4 hours at 80° F.
	Total.	Suspended.	Ammonia N.	Albuminoid N.	Nitrates.	
Eastry (R. S. C., 1902).						
Sewage	1550	1070	25.5	12.8	4.6	123
Bed 1	1460	107	22	3	1.9	50.5
Bed 2	1340	85	12.4	2.4	2.1	25.4
Bed 3	1360	21	4.8	1.2	7.4	17.2
Leeds (Leeds, 1900).						
Sewage	1760	632	27.6	12.4	....	127
Bed 1	1250	274	18.6	7.1	....	62.4
Bed 2	1060	113	13.5	5.1	....	39.6
Bed 3	1030	110	9.7	3.5	2.0	27.5

feet deep and an open space of 33 per cent, which is a liberal estimate, two fillings a day would equal a rate of 650,000 gallons per acre per day and three fillings a rate of about 1,000,000 gallons. In practice, necessary rests and loss of capacity will greatly decrease this amount. At Barking in 1898, Clowes and Houston (1904) obtained with one filling rates of 600,000 for coke and 500,000 for ragstone, and in 1899 with two fillings the rates were increased only to 700,000. Watson (1903) considers

400,000 to 600,000 the best rate attainable, even when the sewage has been previously subjected to septic treatment. The table below, compiled from Watson's Birmingham lecture and from the testimony before the Royal Sewage Commission, indicates the rates which have been obtained in actual operation or in experiments on a practical scale.

TABLE LXIV

CONTACT-FILTER RATES

(Watson, 1903; Martin, 1905.)

Single contact.			Double contact.		
Place.	Depth (feet).	Rate (million gallons per acre per day).	Place.	Depth (feet).	Rate (million gallons per acre per day).
Manchester	3.3	0.6	Burnley	3	0.3
Birmingham	4.5	0.6	Leeds	5.5	0.6
Croydon	3.7	0.8	Blackburn	5.5	0.8
Exeter	5	1	Sheffield	3.3	0.8
Sutton	3.5	1	Carlisle	4	1.1
London	3	1.2	Sheffield	3.3	1.2
Leeds	4.5	1.4			

When a double-contact system is used, the area must naturally be increased, so that the net rate on the total area for a system which will yield a stable effluent will not be over 500,000 gallons per acre per day with three- to four-foot beds.

In general it may be concluded that contact beds will purify sewage at about eight times the rate ordinarily attained by the use of intermittent filters. The effluent will not be so highly purified as in the latter case, but it will be sufficiently stable for all practical purposes. It must be remembered, however, in making such comparisons, that the contact bed has much less flexibility than a sand filter in dealing with exceptionally high flows of sewage. Where large amounts of storm water must be dealt with, special provision must be made in the case of the artificial process.

The Loss of Capacity in Contact Beds. One of the most serious problems in regard to the operation of contact beds is the progressive loss of capacity which appears to be a usual feature of the process. If a bed were filled with perfect spheres of uniform size, its open space or water capacity would be 26 per cent

of its entire cubic contents. In beds built of ordinary irregular materials this original value varies from 30 to 50 per cent. In the course of operation, however, it falls to 20 or even to 10 per cent, and thus reduces the rate of sewage treatment to a half or a quarter of its original value. The rate of capacity loss varies, of course, with the amount of suspended matter in the treated liquor and varies inversely with the size of the filling material. The table below, from the Hamburg experiments, brings out both these points, the fine slag in primary beds showing a greater loss than coarse primary beds or secondary beds of fine material.

TABLE LXV

REDUCTION IN CAPACITY OF HAMBURG FILTERS

(Dunbar and Thumm, 1902.)

Material.	Loss in capacity (gallons per million gallons filtered).	Material.	Loss in capacity (gallons per million gallons filtered).
Single contact.		*Second contact.*	
Slag, $\frac{1}{8}$- to $1\frac{5}{6}$-inch.....	{ 1330 { 1680	Slag, $\frac{1}{8}$- to $\frac{5}{16}$-inch.....	420
		Gravel, $\frac{1}{8}$- to $\frac{5}{16}$-inch.....	700
Coke, $\frac{3}{8}$- to $1\frac{1}{8}$-inch....	340	Coke, $\frac{1}{8}$- to $\frac{5}{16}$-inch.....	630
Gravel, $\frac{3}{8}$- to $1\frac{1}{8}$-inch..	280	Slag, $\frac{3}{8}$- to $1\frac{1}{8}$-inch.....	340
Slag, $\frac{3}{8}$- to $1\frac{1}{8}$-inch.....	170	Coke, $\frac{3}{8}$- to $1\frac{1}{8}$-inch.....	360
Brick, $\frac{3}{8}$- to $1\frac{1}{8}$-inch....	440	Gravel, $\frac{3}{8}$- to $1\frac{1}{8}$-inch.....	460
		Gravel and iron, $\frac{3}{8}$- to $1\frac{1}{8}$-inch	650

The course of this progressive loss of capacity is significant. In the first place, there is an apparent loss, due to the saturation of the surface and pores of the material, which should not properly be included here at all, but which may prove misleading if the original loss of capacity is determined by measuring the fluid required to fill the bed, instead of first filling it and then measuring the effluent. Aside from this false loss of capacity, a true loss begins almost as soon as a bed is put in service, and goes on with considerable rapidity, at a rate of several hundred gallons' loss of capacity for every million gallons of sewage treated. After some six months of use the capacity of the beds decreases much more slowly and may remain fairly constant at perhaps one-half its original value.

The causes of this capacity loss may be grouped conveniently under three main heads: *a*, breaking down and settling together

of material with consequent impairment of drainage; *b*, growth of organic films; *c*, deposition of the insoluble constituents of the sewage.

The decrease in capacity is first, then, due to the breaking down of uniform materials into pieces of more varied size, which become more closely packed together. The amount of this loss may be measured by the space left by the settling over the top of the material. At Pawtucket, R. I., it was estimated that about one-third of the total capacity loss in eighteen months was due to this factor. Such loss may be avoided, to a great extent, by the use of compact and permanent filling. This was shown very clearly in the Hamburg experiments, in which it appeared that slag, while giving as good analytical results as coke and gravel, showed appreciably less loss of capacity. In England a great deal of experience has been accumulated in regard to the settling together and breaking down of friable material. At Leeds Colonel Harding and Mr. Harrison examined the material in a primary bed of coke after one year's use and found that 45 per cent of it passed a $1\frac{1}{2}$-inch screen, although it had originally been over three inches in diameter. At Manchester the rescreening of clinker filling showed that 8–10 per cent had become too fine for use; and at Newton-le-Willows and Andover, after 4–6 years' use, about 25 per cent of the furnace clinker used had been reduced to fine fragments (R. S. C., 1908).

The second loss, due to the growth of the organic films within the bed, is a loss that cannot well be prevented, for it is intimately correlated with the purifying process. As Dr. Fowler has stated: "This (growth of organisms) is at once the cause of increased efficiency in the bed, and of loss of capacity. On examining the material of a contact bed in active condition, every piece is seen to be coated over with a slimy growth. If this is removed, it soon dries to a stiff jelly, which can be cut with a knife. Under the microscope masses of bacteria and zoöglea will be found to be present. If placed in a tube containing air, and connected with a manometer, the jelly will rapidly absorb all the oxygen and produce carbon dioxide" (R. S. C., 1908). Dunbar gives the figures quoted in Table LXVI to illustrate the progressive improvement in the efficiency of contact beds as they grow older and as the surface films increase:

TABLE LXVI

IMPROVEMENT IN AMMONIA ABSORPTION IN A CONTACT FILTER

(Dunbar and Thumm, 1902.)

Months at work.	Reduction in ammonia (per cent).	
	Single filling.	Double filling.
1	9.1	14.6
2	34.6	30.9
5	35.2	23
8	47.4	41.3
10	43	41.2
14	40.5	41.6

The losses due to growth and breaking down of material are almost independent of the character of the liquid filtered. Dunbar treated coke contact filters with various substances (134 fillings in four months) and found with tap water a reduction in capacity from 48 to 40 per cent; with tap water plus 1 per cent urine, from 47 to 37 per cent; with unfiltered sewage, from 48 to 37 per cent; with filtered sewage, from 48 to 40 per cent; and with sewage precipitated with lime, from 44 to 36 per cent (Dunbar and Thumm, 1902).

The loss of capacity due to organic growths can largely be made good by allowing the beds to stand empty for two weeks or more. Under these conditions the organic films dry up, shrivel and disintegrate, and the original capacity of the bed is restored, to a considerable extent, but never wholly. The efficiency of this process is well shown by the results tabulated in Table LXVII, obtained at York and Leeds:

TABLE LXVII

LOSS IN CAPACITY OF CONTACT BEDS AND RECOVERY BY RESTING

NABURN DISPOSAL WORKS, YORK (YORK, 1901)

	U. S. gallons.	Per cent open space.
Bed No. 1.		
Cubic capacity	55,200	100
Initial liquid capacity	22,300	40
After 90 days' work	11,200	20
After 14 days' rest	16,400	30
After 42 days' work	11,500	21

KNOSTROP SEWAGE WORKS, LEEDS (LEEDS, 1900)

Bed No. 7, single contact.		
Cubic capacity	222,000	100
Initial liquid capacity	66,800	31
After 226 days' work	25,900	12
After 74 days' rest	64,200	30
After 184 days' work	30,700	14
Bed No. 8, single contact.		
Cubic capacity	113,000	100
Initial liquid capacity	35,400	31
After 185 days' work	12,800	11
After 50 days' rest	32,300	28
After 203 days' work	11,800	10

There remains, of course, the loss of capacity due to the deposition of suspended solids from the sewage and the change of soluble into insoluble colloids; and this is in some respects the most serious factor of all. It cannot be entirely avoided; and it can only be minified by removing as much as possible of the suspended matter by some preliminary treatment. The relation between capacity loss and suspended solids is well shown in the table below from results tabulated by the English Royal Commission:

TABLE LXVIII

LOSS OF CAPACITY IN CONTACT BEDS

(R. S. C., 1908.)

Place.	Suspended matter in applied liquor, parts per million.	Age of bed, months	Average daily fillings.	Total million gallons treated.	Capacity loss, gallons.	Capacity loss, gallons per million gallons filtered.
Guildford	159	46	2	224	108,794	486
Exeter (main works)	125	37	1.6	2056	750,750	365
Andover	111	38	1.2	96	79,600	828
Exeter (old works)	82	109	1.6	196	39,670	202
Slaithwaite	71	84	2	226	33,755	149

With the exception of Andover, the loss of capacity per million gallons was approximately proportioned to the amount of suspended matter applied to the beds. The high rate of loss at Andover was accounted for by the fact that the material in the beds disintegrated so badly as to cause a considerable sinking of the surface level.

These considerations have led English engineers to make unusual efforts to remove as much suspended matter as possible, before contact treatment, by sedimentation or chemical treatment. The purification of crude sewage is rarely attempted; but septic or chemical effluent may be handled with reasonably good results. In the Barking experiments (Clowes and Houston, 1904), the capacity of two primary coke beds fell in ten months with crude sewage from 69–70 per cent to 18–20 per cent. Secondary beds showed only a reduction from 62 to 55 per cent (coarse) and from 53 to 44 per cent (fine). The stone beds lost about 1 per cent of their original liquid capacity per week. A series of experiments with septic effluent followed, in which after the first loss a capacity of about 30 per cent was constantly maintained. At Leeds it was found that beds taking septic effluent showed much higher capacities than those which received crude sewage. Similar conclusions were drawn by the experts at Manchester, although the experiments made with crude sewage were not exhaustive. The capacity of beds treating septic effluent decreased during the first three months and then remained fairly constant at about 33 per cent. At Burnley, with septic effluent, the capacity of contact beds fell from 44 to 19 per cent; at Exeter it fell from 39 to 28 per cent; and at Leicester from 49 to 29 per cent. At Sutton a minimum of 21 per cent was reached (R. S. C., 1902).

It was hoped that by careful preliminary treatment for the removal of suspended solids, and by resting to disintegrate organic growth, contact beds might be used more or less indefinitely, with a capacity of 20–30 per cent of their original cubic contents. Dunbar and the other German investigators from the first took the view that renewal was inevitable, and that it might be economical not to attempt to keep clogging down to a minimum, but to treat crude or roughly clarified sewage on material as fine as seemed desirable, removing and washing the contents of the bed whenever the capacity fell to 20–25 per cent. This might be required two or three times a year (Dunbar and Thumm, 1902). The Prussian commission at Berlin came to a similar conclusion (Bruch, 1899).

Experience in the main has shown that even with well clarified tank effluents contact beds require renewal after a comparatively short period of time unless the filling materials be relatively

coarse. At Manchester the life of the beds has proved to be about five years. The simple plant used for washing the material at Hamburg is shown in Fig. 81. Similar arrangements for hand washing are in use in many of the smaller English towns. At Manchester the process is more elaborate, being conducted on a large scale. " The method employed is to pass the material from a sump to a jigging-screen of $\frac{1}{4}$-inch mesh, over which are fixed a series of horizontal water sprayers. The material rejected by this screen is further graded by passing over a fixed

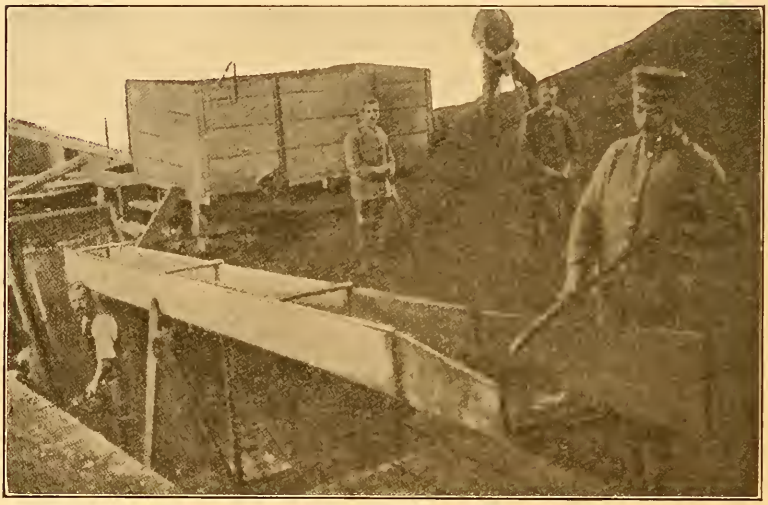

Fig. 81. Washing Contact Material by Hand at Hamburg (copied by permission from Dunbar, 1908).

2-inch mesh screen. The material which passes through the $\frac{1}{4}$-inch screen falls onto an inclined fixed screen of $\frac{1}{8}$-inch mesh. All material above $\frac{1}{4}$-inch is replaced in the primary filters; that between $\frac{1}{4}$-inch and $\frac{1}{8}$-inch will be used for the surface of the secondary contact beds, when these are constructed. Settled sewage is the liquid used in the washing. The total cost of removing, washing, screening and replacing in beds and making up to original level with new material at Manchester is 1*s.* 6*d.* per cubic yard " (R. S. C., 1908).

In general the cost of washing and renewal at the English contact plants has ranged between 25 and 50 cents per cubic

yard. The material lost amounts to 20–25 per cent of the original filling; but the washed material is better than it was at first, as the softer and more friable portions have been eliminated.

The washing of the contact bed material at Plainfield has cost about 60 cents per cubic yard.

At Canton and Alliance, Ohio, the sewage is discharged into the contact beds through a body of gravel held in place in a wooden conduit for the purpose of concentrating the clogging in the gravel, which can be easily handled and cleaned. In fact, at Canton, provision is made for washing the gravel on a permanent shelf provided with water under pressure and also with drainage to carry away the dirty wash water. According to the experience of the writers it will often be unnecessary to remove and clean contact bed material on account of clogging if the material be large (1.0–1.5 inches in diameter) and the beds carefully operated.

Experience with Contact Beds at Manchester and Other English Cities. Until 1889 all the sewage of Manchester was discharged untreated into the four streams and rivers that flow through the city, finding its way ultimately into the Irwell. In that year Parliamentary power was obtained to divert and treat the sewage, and 95½ acres of land, subsequently increased to 165½ acres, were acquired at Davyhulme, about five miles from Manchester, for the erection of works. The scheme involved 36½ miles of intercepting and other sewers, a system of eleven tanks with a total capacity of 12,000,000 gallons for chemical treatment, a plan for dealing with the resulting sludge by eight filter presses, pumping engines, sludge wells, etc., and 36 acres of land for further treatment of the effluent from the precipitation tanks.

The works were completed in 1893. The sewage as it came to the plant was screened, lime and sulphate of iron were added, and the liquid was then passed through the precipitation tanks. It was the original intention to discharge this effluent on the 36 acres of underdrained land, but this was never actually done, and the effluent, or at least the greater part of it, was discharged directly into the ship canal. The result was unsatisfactory from the very first; the effluent obtained from the chemical treatment would not answer the requirements of the English provisional standard, and underwent secondary decomposition.

Chemical precipitation for the treatment of Manchester sewage having thus been shown to be unsatisfactory, it was suggested

that the effluent from the precipitation plant be purified by irrigation; but as this would have required 1300 acres, exclusive of the roads, carriers and the usual adjuncts of a sewage farm, it never received very favorable consideration.

In 1895 Sir Henry Roscoe suggested that the effluent from the precipitation tanks should be purified by treatment on bacterial contact beds, similar to those tested experimentally by Dibdin at Barking; and by his advice two small bacteria beds, afterwards increased to four, were built, and an elaborate series of experiments were undertaken under the immediate direction of Frank Scudder. The beds were 18 feet long and 12 feet 6 inches wide and 4 feet deep, and were filled to the depth of 3 feet with the filling material, — one with coke, one with cinders, one with burnt clay and one with coal. The result of these experiments was, briefly, that cinders, on the whole, were found to be the best material, and that an effluent answering to the English standard, and one that would not undergo subsequent putrefaction, could be obtained by treating the effluent from the precipitation process at the rate of about 700,000 gallons per acre per day on prepared bacterial single-contact beds.

While the experiments under Mr. Scudder were still being carried on, Mr. Meade, the city surveyor, though expressing his confidence in the results that had been obtained, presented to the Committee on Drainage, March 1, 1896, a plan for carrying the effluent from the precipitation tanks $15\frac{1}{2}$ miles in a covered conduit to the tidewaters of the Mersey at a cost of about $1,300,000 and recommended the plan for adoption as solving for all time the sewage problem. This plan, however, was not accepted by the city and in May, 1898, a commission was appointed, whose large scale experiments, carried on under the direct charge of Gilbert J. Fowler, indicated that both the dry and storm weather flow could be successfully treated by a combination of the septic tank treatment with contact beds, and in October, 1899, the double-contact method was recommended for the treatment of Manchester sewage. Work was begun almost immediately and the first-contact beds were put into service in 1901.

In 1909 the general plan of the sewage plant was as follows: Four settling tanks, capacity of each 1,630,000 gallons; 12 septic tanks, total capacity of 19,500,000 gallons; 92 first-contact beds, each 0.5 acre superficial area, constructed of cement con-

crete and filled with furnace clinker (rejected by a 1.6 inch mesh passed by a 0.25 inch mesh) to a depth of 40 inches. Besides these 92 first-contact beds there was an area of 27 acres divided into 29 storm-water contact beds. In these beds the filling material, 2.5 feet of unscreened furnace clinker, rested generally upon a heavy clay marl, though where needed a layer of cement concrete was laid down. They were designed to operate at a rate of 3,000,000 gallons per acre per 24 hours. The original plan also included 52 half-acre second-contact beds similar in construction to the first-contact beds, except that the filling material was to be somewhat finer.

The amount treated on the primary contact beds has averaged 600,000 gallons per acre per day. The effluent has not, however, been perfectly satisfactory, often undergoing secondary putrefaction. The effluent of the first-contact beds when passed on through the first of the second-contact beds to be built yielded on effluent that was nonputrescible, and it was therefore planned to treat all of the effluent from the first-contact beds on secondary beds. Allowing that a suitable effluent can be thus obtained, it requires for the satisfactory treatment by double contact of 600,000 gallons per day 1.5 acres, or one acre for 400,000 gallons.

The general result at Manchester has justified the original design of the plant, and the works have treated by contact filters 90 per cent of the total flow, with an average purification, as shown below, of over 70 per cent, and have yielded an effluent often less putrefactive than the water of the ship canal.

TABLE LXIX

EFFECT OF TREATMENT OF MANCHESTER SEWAGE ON CONTACT BEDS

Results for the year ending March 30, 1904.

(Parts per million.)

Source of sample.	Ammonia N.	Albuminoid.	Nitrogen as nitrites and nitrates.	Oxygen consumed.
Crude sewage	22.7	5.2	.....	105.4
Septic tank effluent	26.3	3.7	.....	93.5
Primary contact-bed effluent	16.2	1.5	2.7	27.2

Average purification: On albuminoid N basis, 71 per cent.
On oxygen-consumed basis, 73 per cent.

During the first years of the working of the plant, owing largely to the amount of suspended matter in the effluent from the septic tank, Mr. Fowler anticipated that it would be necessary to wash and replace the filling material, and in 1907 the beds were clogged to such a degree that it was considered desirable to remove the filling material, wash it, and, after replacing the loss of material,

Fig. 82. Filling Contact Beds at Manchester.

to refill the beds. This was done at an average expense of about 31 cents per cubic yard.

Cost data of great value have been accumulated at Manchester. The amount of money to be spent for the construction, including the original works for precipitation, aggregated about three million dollars. The first cost of building the contact beds, including excavation, underdraining, laying concrete and putting filling material in place, was $33,000 per acre. The filling material in place cost 87 cents per cubic yard. The cost of removing filling material, washing and replacing it, and making good the

fine material lost in washing, brings the total cost of renewal up to about 50 per cent of the first cost of filling. The general operating costs of the plant are tabulated below from the figures for 1906–1907:

TABLE LXX

WORKING COSTS OF TWELVE PRIMARY CONTACT BEDS AT MANCHESTER

(Eng. News, 1908.)

Average number of fillings per bed	2,690
U. S. gallons septic effluent treated on 12 beds (6 acres)	4,613,624,000
Total maintenance cost	$4,087.26
Total renewal cost (40½ cents per cubic yard)	$13,705.20
Maintenance cost per million gallons filtered	$0.89
Renewal cost per million gallons filtered	$2.97
Actuating valves per million gallons filtered	0.25
Total working cost per million gallons filtered	$4.11

The average per capita cost of sewage treatment for the five years 1903–1907, including sludge disposal, filtration, all maintenance and renewals and all general expenses, but with certain receipts deducted, varied from 12 to 18 cents.

About 1900 a number of other contact systems were installed or recommended in English cities. Huddersfield had a serious problem in the presence of large amounts of industrial waste from the scouring and dyeing of wool; but it was shown in a series of experiments carried out between 1898 and 1900 by J. L. Campbell that chemical treatment, sedimentation and contact treatment would solve the difficulty satisfactorily (R. S. C., 1902). At Oldham studies carried out by J. B. Wilkinson from 1898 to 1900 led to the adoption of sedimentation and single-contact beds (R. S. C., 1902).

A valuable review of existing conditions in 1904 by M. N. Baker (1904) described septic tank and contact filter plants in operation at Exeter, Yeovil, Barrhead and Burnley, as well as at Sutton, Manchester and Oldham. In general, the tendency of late has been away from the construction of contact beds, — on account of their short life, and in view of the higher rates attained with trickling filters.

At Sheffield, however, a city of nearly half a million population, with a sewage flow of fifteen million gallons, a system of single-contact treatment was installed in 1908. The plant is thus described by H. W. Clark (1908):

"Sixteen settling tanks are being constructed, each with a capacity of 1,000,000 gallons, and chemical treatment is to be omitted. Thirty acres of contact beds in half-acre sections are being constructed. All these contact beds are most solidly built with brick wall, concrete bottoms 6 inches thick and brick and concrete channels. The beds are to contain 4 feet in depth of clinker over the underdrains, and the main underdrains are being built of concrete below the floor of the filter with tile coverings, and side drains 10 feet apart entering these are laid on the concrete flooring. The material of the bed is to be graded clinker 3 to 6 inches in diameter and becoming finer towards the top, the upper six inches to be constructed of clinker not more than $\frac{1}{4}$ or $\frac{3}{8}$ inches in diameter. The sewage is to pass to these contact beds through a channel built between each set and will enter the bed through a 2-foot pipe to a chamber in the center of the bed, where it will rise and overflow to a second circular chamber 15 feet in diameter. From this it will pass over the surface of the beds in channels formed of the fine surface coke. The building of contact filters at Sheffield is a result of the operation, for ten years, of large experimental contact filters treating 1,000,000 gallons of sewage daily. These filters produced an effluent equal to the requirements of the Local Government Board, and it is stated that the filtering material was never cleaned or renewed during their period of operation."

Contact Beds in the United States. After the adoption of contact beds by Manchester in 1900, many plants of this type were built in the United States, particularly in the middle-western states. There are at present some forty such plants in operation in tne United States, of which 24 are in New Jersey (Daniels, 1914) and perhaps a dozen in Ohio.

The contact installation at .Plainfield, N. J., was designed by J. O. Osgood, with Andrew J. Gavett in charge of construction. The population of this city is about 20,000 and the sewage flow about 1,000,000 gallons a day. Intermittent filtration was at first adopted, but the beds used were of too fine grain and clogged badly. In 1900 a septic tank and double contact filter system was substituted, and in 1905 the plant was considerably enlarged to meet the increased sewage flow.

The sixteen contact beds are of concrete construction, each 92 by 106 feet in area and 5 feet deep. The eight secondary beds are 5.5 feet lower than the primary beds, but of the same dimensions. On the floor of each bed fourteen lines of 4-inch horseshoe draintile were laid, converging to a controlling gate chamber at

the center of each group of four filters. Over and between these tiles was placed a 6-inch layer of coarse broken stone upon which rested the main body of the filling material. This material was at first between $\frac{1}{4}$ inch and 1 inch in size for the primary beds and $\frac{1}{8}$ inch to $\frac{3}{4}$ inch for the secondary beds, but when reconstructed in 1905 all the beds were filled with $\frac{1}{2}$ inch to 1 inch material. These beds have from time to time given a great deal of trouble from clogging in spite of the fact that they receive clarified sewage.

The Plainfield plant is operated entirely by hand. Of the eight contact beds in each set, six are used in regular rotation and the other two held in reserve. The ordinary cycle is a 12-hour one, two hours each for filling, standing full and emptying, and six hours for rest. Of the results the report of the State Sewerage Commission (N. J., 1907) is as follows: "The effluent from the primaries is almost clear. Its odor is musty but not offensive. The final effluent is clear but not sparkling. Its odor, when an outlet gate is first opened, indicates the presence of considerable free ammonia. As the bed empties, this disappears and an earthy odor takes its place. The drainings are almost odorless. The effluent is an excellent double contact filtrate effluent easily assimilable by the stream which receives it."

The following analytical data are quoted by Fuller (1909) for the period from September, 1908, to March, 1909:

TABLE LXXI

RESULTS OF OPERATION OF CONTACT BEDS AT PLAINFIELD, N. J.

September, 1908–March, 1909. (Fuller, 1909.)

	Parts per million.			Putrescibility.	Bacteria per c.c.
	Suspended solids.	Oxygen consumed.	Nitrogen as nitrates and nitrites.		
Screened sewage	117	86	.....	....	2,370,000
Septic sewage	53	51	.....	....	1,150,000
Primary effluent	19	22	0.9	+	810,000
Secondary effluent	11	11	4.4	0	470,000

Numerous small contact beds have been built in the West, but many of them, through want of careful supervision, have not proved a success. The chief trouble has been caused by mismanagement and by dosing the beds with septic tank effluent which contained a large amount of suspended matter. The beds

have thus become quickly clogged and have been unable to take the amount of sewage required. The effluent from these contact beds has, however, been generally nonputrescible.

One of the largest of the contact bed plants installed in the West was built by Snow and Barbour at Mansfield, Ohio. Mansfield is a manufacturing center with a population of about 25,000, and the dry-weather flow is in the neighborhood of 1,000,000 gallons a day. The sewage was originally passed through a grit chamber fitted with an inclined screen 10 feet by $5\frac{1}{2}$ feet in area, made of $\frac{3}{4}$-inch wrought iron bars, set $1\frac{1}{2}$ inches apart. Over the screen chamber the sewage passed to four septic tanks. These tanks were of stone masonry and of concrete, built

FIG. 83. General View of Contact Beds at Mansfield, Ohio.

underground and covered with turf, so that nothing was visible except a slight elevation of land. Each of the four tanks was 52 feet by 92 feet 3 inches in area, and 9 feet 6 inches high, the normal water line being 7 feet above the bottom.

The contact beds were five in number and laid out in the form of a circle, of which each unit formed a sector, with a regulator house at the center. The total area was an acre and a quarter. The beds were principally in excavation, but partly in fill, and the walls were of natural excavated clay. The filters were filled to a depth of 4 feet 9 inches with $\frac{1}{4}$- to $\frac{3}{4}$-inch crushed cinders, each bed being very thoroughly underdrained with open-joint tile pipe. At the end of each of the seven main branch underdrains was a 4-inch iron ventilating pipe. Distribution on the surface of the bed was effected by branched wooden carriers with hinged joints at each change of section. The arrangement of these carriers, with a general view of the beds themselves, is shown in Fig. 83.

The dosing of the contact beds was regulated by an ingenious patented automatic device. The sewage flowed over aerating steps and thence to a cylindrical chamber under the gatehouse at the center of the contact area. Radiating from this chamber were five 8-inch pipes leading to the five beds. The flow of sewage was directed on to any one filter by a revolving gate, cylindrical in shape, concentric with the chamber and machine fitted to its interior. This gate had a single port, which could be brought by its revolution opposite the inlet of any one of the beds. Below

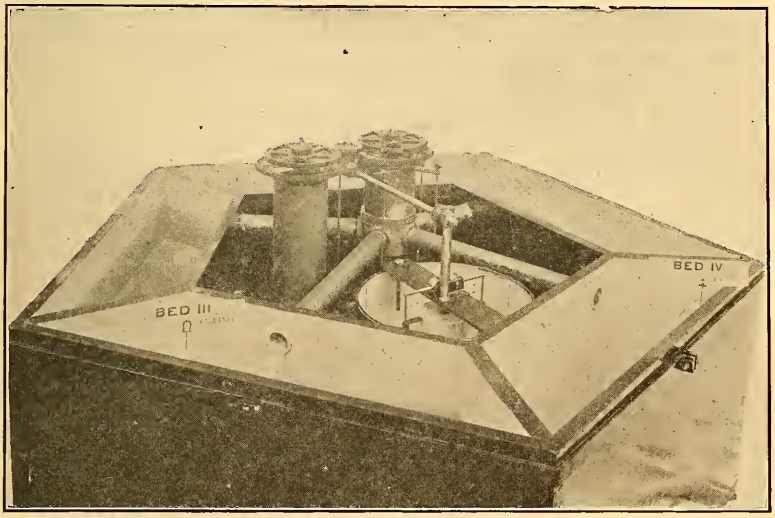

Fig. 84. Model of Dosing Device for Contact Beds (courtesy of F. A. Barbour).

the influent chamber was a similar cylindrical chamber which received the effluent from the filters through radiating pipes at the sides and discharged it through a bottom opening. The movement of the revolving gates in the two chambers was controlled by a float, which intermittently rose and fell, and in so doing, operated the cylinders through a train of gears. The float, in turn, was actuated by the rise of sewage in the bed which happened to be filling. The complete filling of the float chamber was adjusted to a one-fifth revolution of the cylindrical gates, and the opening of the outlet from a given filter was arranged to follow two-fifths of a revolution after the opening of its inlet. Filling

and standing full occupied about $2\frac{1}{4}$ hours, and the empty period was five hours. This automatic apparatus operated with marked success. Figure 84 is from a photograph furnished by F. A. Barbour, which illustrates the working of a model prepared to show the general principles involved. When the float chamber fills, the inlet of bed *III* closes, the outlet of *II* opens and the sewage flow is diverted to *IV*. Between the cylinders are seen the cogs by which the motion is transmitted.

The septic tanks worked very unsatisfactorily, the effluent sometimes containing more suspended matter than the sewage, and this has led to very serious clogging of the beds; analytical data for a 24-hour test in May, 1907, are tabulated below:

TABLE LXXII

EFFICIENCY OF SEPTIC TANKS AND CONTACT BEDS AT MANSFIELD, OHIO

Parts per million. (Kimberly, 1908.)

	Oxygen consumed.		Nitrogen as —					Suspended solids.		Dissolved oxygen.	Fats.
			Organic.								
	Total.	Dissolved.	Total.	Dissolved.	Ammonia N.	Nitrite.	Nitrate.	Total.	Volatile.		
Sewage.....	37	25	11.5	7.6	7.8	0.2	0.4	74	55	2.3	43
Septic effluent....	32	.....	12.2	....	9.4	0.05	....	110	41	2.5	....
Contact effluent..	9.2	.....	0.8	....	3.6	0.04	2.6	40	1	2.5	....

The original cost of the Mansfield plant included $8000 for land, $15,870 for the construction of the septic tank and $19,850 for the construction of the contact filters. The cinders used for filling the beds cost 85 cents per cubic yard in place. The cost of operation was $5644 in 1906 and $5260 in 1907, amounting to 47 and 44 cents per person contributing sewage; but more than half of this cost is for pumping.

The Mansfield plant was originally put in service in 1902 with a dry-weather flow of 765,000 gallons of weak sewage a day. By 1914 the flow had increased to 1,899,000 gallons and the strength to a normal figure. The capacity of the plant was therefore far outgrown, raw sewage was at times discharged into the river, and

vigorous complaints were made by riparian owners. Mr. Barbour, the designer of the original plant, recommended that the rate of contact treatment be reduced to 600,000 gallons a day and that Imhoff tanks and trickling filters be constructed to deal with the excess flow.

Dibdin's Slate Beds. Mr. W. J. Dibdin, the originator of the contact bed, has devised a modification of it which is so radical as to deserve particular discussion. This is the slate bed. A firm believer in the contact system, Mr. Dibdin was confronted by the loss of capacity due to progressive clogging and the consequent necessity for washing and renewal of the filtering material. He was opposed to the use of the septic tank or any anaerobic process; some means for treating the crude sewage in aerobic beds was what he sought. He reasoned that an increase in the air content of the contact bed would not only increase the liquid capacity of the bed, but also allow sufficient aerobic action, so that the solid matter left in the bed would be much less in quantity and much less decomposable than the suspended matter in the sewage. After various experiments he decided that if a contact bed was filled with layers of slate slabs separated by one-fourth to one-half inch spaces, very much better results would be brought about, particularly if the slate were so laid that when necessary the solid matter deposited or remaining in the bed could be washed out while the slate was *in situ*.

After twelve months' laboratory work an experimental bed was constructed at Devizes in January, 1904, and this proved so successful that the process was adopted for the whole of the sewage of the town, the beds coming into operation on the 12th of September, 1905. Since then the system has been installed in some fifty different places, in many cases displacing septic tanks, chemical treatment processes, etc. It has received the sanction of the Local Government Board for Ireland for the treatment of three million gallons per diem at the Sydenham outfall site of the main drainage system of Belfast, and it was installed for the War Office at the Cunagh Camp, Ireland. The general construction of a slate bed is indicated in Fig. 85.

In the Devizes experimental beds the initial working capacity was 82 per cent of the total cubic content of the bed. After 14 months this was reduced to 50 per cent; the opening of the outlet valve to its full extent, so as to permit the beds to empty with a

rush, increased the value to 64 per cent, and the removal of some of the slates at one side, so as to permit the slates to be flushed out with a hose, restored the original capacity of 82 per cent.

The plant at Devizes was examined by A. C. Carter (1909) for the Royal Commission on Sewage Disposal, and the results of his examination may be summarized as follows: It is to be

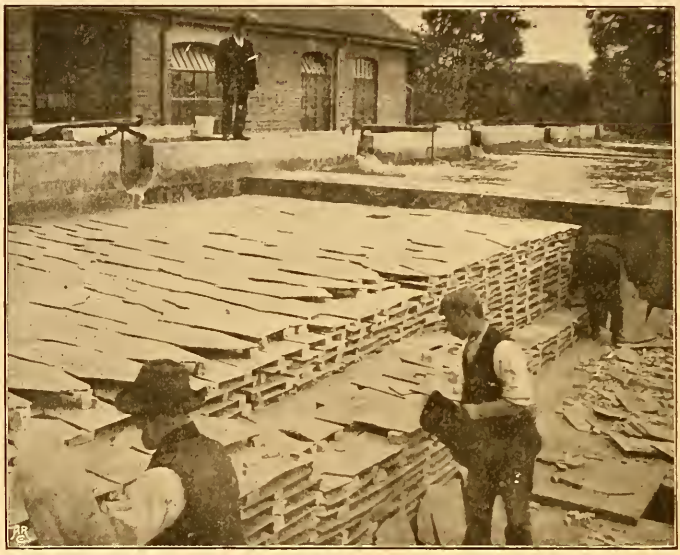

Fig. 85. Filling Slate Beds at Devizes (courtesy of W. J. Dibdin).

noted that this examination was made in the month of April, when the winter accumulation of solid matters is worked off at its most rapid rate.

The dry-weather flow of sewage is about 200,000 gallons per day, and the installation consists of 9 slate beds for preliminary treatment and 8 fine coke beds for final treatment. Seven of the slate beds are 45 feet by 68 feet by 4 feet deep, for dealing with the dry-weather flow; 2 are 80 feet by 68 feet by 4 feet, for dealing with storm water. They are constructed of superimposed layers of slate separated by pieces of slate blocks 2 inches to $2\frac{1}{2}$ inches thick. At the bottom of the bed is an open space of about 6 inches (all later beds have only a 2-inch space) and the slate is built up to an average height of 3 feet 6 inches. The beds are usually filled to a depth of three feet. The calculated capacity

of the smaller beds is 60,000 U. S. gallons, and that of the larger beds 108,000 gallons each.

Crude unscreened sewage is run directly on to the beds, requiring three hours for filling when the flow is normal. The sewage then stands in the beds in a quiescent state for two hours, when they are emptied, this taking about three hours. When the outlet valve is opened, a considerable amount of black suspended matter comes away with the liquid. Average samples of the first ten minutes' flow gave 304 parts per million. In times of storm the sewage is passed through a detritus tank to catch road grit before running on the slates. Daily analyses of composite samples of the sewage and effluent were made for one week with the following results. The effluent samples did not contain any of the suspended matters which come away during the first ten minutes.

TABLE LXXIII

ANALYSES OF SEWAGE AND EFFLUENT AT DEVIZES

Parts per million. (Carter, 1909.)

	Number of samples.	Crude unscreened sewage.	Slate bed effluent.	Percentage reduction.
Suspended solids	7	446	149	66.6
Volatile matter in these solids	7	346	125	63.9
Solids by centrifuge (vols.)	7	2667	849	68.1
Ammonia N	5	63	70.3	.....
Albuminoid N	5	15.7	11.6	26.1
Organic nitrogen	5	35.4	24.9	29.6
Total nitrogen	7	109.9	91.9	16.4
Oxygen absorbed in 4 hours at 27° C. from N/8 permanganate	7	215	133	38.1
Chlorine	.....	201	181	.....

From results given in the above table it was calculated that the amount of dry solid matter in the sewage amounted to 145 tons per annum. The amount in the effluent was 48.5 tons, leaving 96.8 tons to be dealt with on the beds. The amount of dry solid matter discharged from the beds as humus, calculated from one day's experiment, equalled 41 tons per annum, leaving 55.8 tons either digested or retained by the beds. It should be taken into consideration that the above results are calculated from data obtained during a single week in April, and may not therefore be typical. The deposit on the slates in the bed was examined

by opening the bed at one side down to the bottom. It averaged about ½ inch in thickness and was full of small red worms. No offensive odors could be detected, the smell being exactly that of a damp cellar.

Experiments at Philadelphia have shown that the slate bed works well as a preliminary process with American sewage. Three-fourths of the suspended matter in the sewage was deposited, and the deposit was earthy and inodorous (Philadelphia, 1911).

It will be noted that the characteristic feature of the slate bed is that it is a device for reducing solids in suspension. It is in many respects comparable with a sedimentation tank rather than a sewage filter. So far as the sewage is concerned the slate bed is really a series of small sedimentation tanks. The sludge deposited is, however, reduced under aerobic conditions and not under anaerobic conditions as in the septic tank; and the suspended matter which passes off in the effluent is in a stable humus-like form. The decrease in the dissolved organic matters is less than in the ordinary contact bed, although sufficient to render the effluent suitable for treatment on secondary beds or direct discharge into tidal estuaries, or on to land. The working capacity is double that of the ordinary clinker bed, and thus one-half of the construction cost of the tanks is saved. After nearly five years' work the English beds showed no signs of requiring cleaning, but continued to maintain a good working capacity without the depth of deposit increasing.

One of the most interesting points about the slate beds is the prominent part played by the higher forms of organisms. All observers note the immense numbers of worms, insects, crustacea and arachnida present in addition to protozoa and bacteria. Dibdin (1909) believes that the activity of the slate bed is in large degree a digestion of the solid matter carried out by these higher organisms; and the same thing is probably true to a less extent of the trickling bed, which frequently contains enormous numbers of worms.

The Peculiar Sewage Problem at Belfast, Ireland. At Belfast the disposal of sewage is complicated by an unusual condition, which makes sand or trickling filters unsatisfactory; and the contact bed has offered just the necessary solution of the difficulty. As a rule, oxidation is the ultimate aim of sewage purification.

At Belfast, however, the main condition which has made sewage purification urgent has been the heavy growth of seaweed in the harbor. The sea lettuce, *Ulva latissima*, and other Algæ flourish in such enormous quantity in Belfast Lough, that when the masses of seaweed are washed ashore and putrefy they cause a nuisance of grave extent. A study of the distribution of the seaweed in relation to the pollution of the harbor made it clear that the excessive growths were due to the fertilizing action of the nitrogen of the sewage. Such green plants can utilize unoxidized nitrogen; but they can also utilize nitrogen in the form of nitrates and nitrites. So, even the complete nitrification of this sewage would not suffice. The actual removal of some considerable portion of the nitrogen seems to be the only way out of the difficulty.

Extensive investigations carried out by Dr. E. A. Letts have shown that treatment of crude Belfast sewage in double contact beds would remove 51 per cent of the ammonia nitrogen, and 71 per cent of albuminoid nitrogen and oxygen consumed, with but little formation of nitrates. Combined septic and double contact treatment gave better results — 87 per cent purification measured by ammonia nitrogen, 73 per cent measured by albuminoid nitrogen and 78 per cent measured by oxygen consumed. In this case, however, 6.2 parts per million of nitrogen as nitrates appeared in the effluent. Treatment by a septic tank and trickling filter gave still better purification with one-third the filter area; but the nitrate formation was naturally much greater than with the contact bed.

Trickling beds seemed therefore to offer the best method for purification, if the nitrates could somehow be removed. Dr. Letts, therefore, hit on the expedient of supplementing the trickling process by secondary denitrifying beds on the contact plan. He found by mixing a nitrate solution with septic effluent that 25 parts of nitrogen per million disappeared in 24 hours. A small contact bed was constructed containing $1\frac{1}{2}$ feet of $\frac{1}{4}$-inch broken brick. This was dosed with mixtures of various proportions of septic effluent and trickling filter effluent giving the general results cited in Table LXXIV.

TABLE LXXIV

RESULTS OF SEPTIC, TRICKLING AND DENITRIFYING TREATMENT AT BELFAST

Parts per million. (Letts, 1908.)

	Septic effluent.	Trickling effluent.	Denitrified effluent.	Per cent purification.
Nitrogen as ammonia nitrogen	32	9.6	14.3	61
Nitrogen as albuminoid nitrogen ..	6.8	3.4	3.7	46
Nitrogen as nitrates.....	.....	16.6	1.7	.....
Oxygen consumed.....	65.3	24.5	23.1	57

By using a larger proportion of trickling effluent the purification in organic constituents could be improved to over 80 per cent but a larger amount of nitrates would be left. The method finally recommended is as follows (Letts, 1908):

" After screening, and the removal of road detritus, the sewage should be submitted to a process of sedimentation in suitable tanks for a period of six hours, then a portion thus clarified should be treated on sprinkling filters, after which the resulting effluent should be mixed with the remainder of the tank effluent, and the mixture submitted to further treatment in a denitrifying bed, then discharged into ponds or lagoons, and thence into the Lough. Provisionally, a mixture of equal volumes of the tank and sprinkler effluents may be treated on the denitrifying beds, and the working cycle of the latter may be 4 hours' contact and 2 hours' rest; but it is quite possible that future experience may show that these conditions may be so modified as to induce a more complete purification than that hitherto obtained, which amounts to about 80 per cent in those factors which affect the growth of the *Ulva*."

The main point of interest in the investigations is the light they throw on the special value of the contact system in the elimination, as distinguished from the mere oxidation, of nitrogen. The Belfast problem is a rare but not a unique one; and wherever the actual removal of nitrogen is for any reason desirable, the contact bed offers a peculiarly satisfactory method of treatment.

Cost of Contact Treatment. Careful estimates made at Manchester, England, quoted above show, a total maintenance cost of $4.92 per million U. S. gallons ($4.11 per million Imperial gallons), of which $3.57 per million U. S. gallons was for renewals.

Costs of $0.47 and $0.44 per person contributing sewage have been cited for Manchester, one-half of this amount being for pumping. For Plainfield, Fuller gives the costs tabulated on p. 314 for the entire plant (septic tanks and double-contact beds).

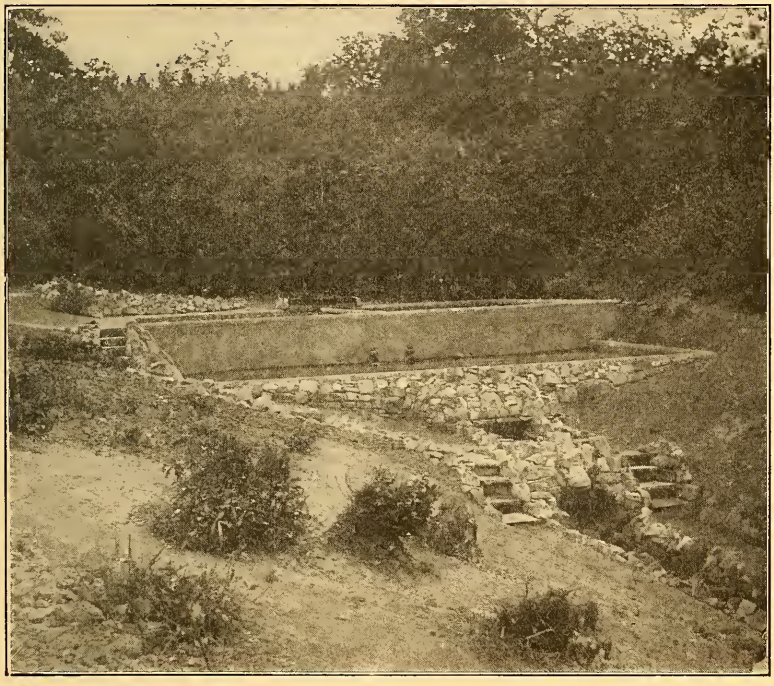

Fig. 86. Small Contact Installation in England.

This would give an average cost for the four-year period of about $4400, of which somewhat over one-fourth was for the manager-chemist and consulting engineer, a little less than one-fourth for the care of the contact beds, and a little less than one-sixth for renewal of contact beds. On a flow of 1,800,000 gallons a day the total average cost for the four years amounts to $6.70 per million gallons treated.

TABLE LXXV

OPERATING COSTS, PLAINFIELD, N. J.

	1907.	1908.	1909.	1910.
Manager-chemist, consulting engineers	.....	$1325.50	$1818.46	$1677.67
Care of contact beds	$1180.53	1189.26	885.09	918.68
Improvement of contact beds	.....	.....	2032.87	935.36
Care of septic tanks	662.25	603.50	252.89	269.17
Repair of septic tanks	101.14	.....	151.89	1011.15
Other expenses	864.21	696.44	579.89	424.28
Total expenses	$2808.13	$3814.70	$5721.09	$5236.31

Advantages and Disadvantages of Contact Treatment. The contact system is well established as one of the satisfactory methods of sewage disposal. Double contact will yield a stable effluent of sufficiently good character to be discharged into almost any stream. By comparison with the trickling filter, however, the contact bed is in general distinctly an inferior process from the standpoint of cost efficiency. Comparative estimates for the two processes made by both English and American engineers will be quoted in the following chapter. The distributing system for the trickling bed is costly, both in construction and in operation; but the area required for trickling filters is only from one-third to one-half that needed for contact beds.

For certain special cases, however, the contact bed is better suited than the trickling filter. Its effluent differs from that of the trickling filter in two important respects. It contains only a small proportion of mineral nitrogen and is comparatively free from suspended solids. It will therefore meet the requirements of a peculiar case like that of Belfast; and it will often serve for plants where the disposal of the suspended solids in the trickling effluent might prove to be a burden. Another distinct advantage of the contact bed under certain conditions is the low head under which it can be operated. A trickling filter requires at the least 8 feet of head for the bed itself and for the distributing apparatus; while a double contact bed could, if necessary, be crowded into 5 feet. Altogether a contact installation lends itself to compact and inconspicuous construction, which is of much practical importance in the design of small plants for institutions or for private houses. The contact bed produces less odor than the trickling

filter and does not breed flies as the trickling filter does. It may therefore safely be installed much nearer to dwellings. Another advantage in the contact system, for small disposal plants, lies in the fact that it adapts itself more readily to marked irregularities of flow than does the trickling bed. If, however, plants of this type are designed to work under the control of automatic apparatus, it must be remembered that a lack of occasional supervision is likely to lead to disastrous results.

CHAPTER XI

TREATMENT OF SEWAGE IN TRICKLING OR PERCOLATING BEDS

Historical Development of the Trickling or Percolating Filter. At about the same time that Dibdin was working out the principles of contact treatment, several engineers in England and America were laying the foundation for another method of purification, by rapid filtration through coarse materials, on purely aerobic lines. As at Barking, the reduction of the filtration area necessary for sand filters was the chief end in view. Experiments at Lawrence on the filtration of sewage " through clean gravelstones larger than robins' eggs " had already furnished the first suggestion of such a process. In 1892 Hazen started a filter of one-fifth inch material which received four doses of sewage a day and was artificially aerated. The rate was increased from 140,000, at the start, to 480,000 gallons per acre per day. The surface clogged badly, but the effluent was good, showing 30 parts per million of nitrates. This filter was dosed with an automatic siphon; but it was clear that in using coarse material some device must be introduced to secure a rather slow and regular passage of sewage through the bed. The first method adopted for this purpose was the spreading over the surface of a thin layer of finer material; but this closed the surface of the bed and required forced aeration for the maintenance of aerobic conditions. In 1892 Lowcock, at Malvern, England, constructed a gravel filter with a sand layer on its surface and filtered chemical effluent at a rate of nearly 300,000 gallons per acre per day, forcing air under pressure into the middle layer of the bed. A good effluent was obtained and the filter was operated for fifty-one days without rest (Lowcock, 1894). Similar filters were later constructed at Wolverhampton and at Tipton (Rideal, 1901). At both places ordinary trickling filters have since been installed. In the United States, Waring was attempting at the same time to use the principle of forced aeration. He obtained a patent on his process as early as 1891, and carried out a series of experiments at Newport in 1894 on " the me-

chanical straining out of all solid matters carried in suspension in sewage and their subsequent destruction by forced aeration and the purification of the clarified sewage by bacterial oxidation of its dissolved organic matter in an artificially aerated filter." The sewage, in Waring's words, " trickles down in a thin film over the surfaces of the particles of coke or other filtering material, while through the voids between the particles and in immediate contact with the trickling films of liquid a current of air is con-

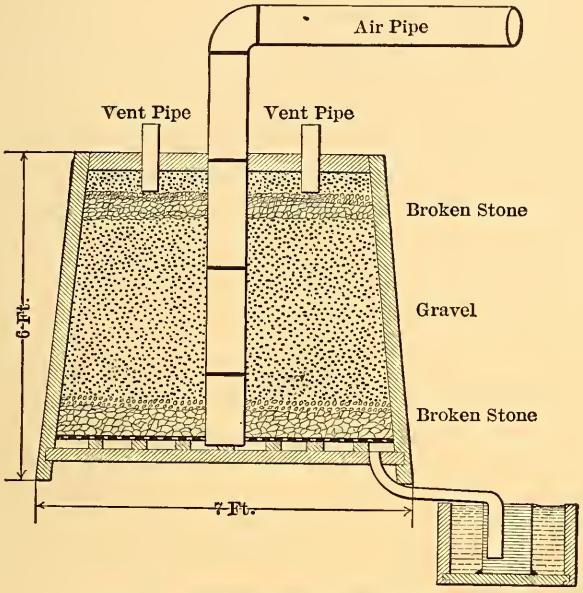

Fig. 87. Diagram of the Waring Filter.

stantly rising, being introduced at the bottom of the tank by a blower." Fine gravel on the surface of the main body of filtering material effected the even distribution of the sewage, and the grosser suspended solids were removed by preliminary straining through broken stone. It is stated in the report of these experiments that the aerators removed " over 95 per cent of the organic nitrogen of a strainer effluent applied at a rate of at least 800,000 gallons per acre per day " (Waring, 1895).

Waring's principle of oxidation was undoubtedly correct; but the plants actually installed on the Waring plan have not generally operated with marked success. A good example of the

process could be seen for a time at East Cleveland, Ohio. This plant included 0.112 acre of primary strainers, 0.056 acre of secondary strainers and 0.248 acre of aerator-beds and was designed for a sewage flow of 300,000 gallons a day. This rate was almost doubled, as the result of an increased number of connections; and under these severe conditions the beds — strainers and aerators alike — became badly clogged and the effluent was imperfectly purified. Extensions of the Cleveland sewerage system finally led to the abandonment of the plant.

A practically successful solution of the problem of rapid filtration under aerobic conditions was finally reached along another line, by abandoning the application of sewage in bulk, with artificial aeration of the filter from below, and resorting instead to the device of applying the sewage continuously, or at very frequent intervals, in a fine spray distributed evenly over the whole surface of the bed. By this means the too rapid flow of large currents of sewage is prevented, and at the same time air is drawn in for oxidation from the top and sides of the filter. Sewage applied in this way trickles in thin films over the surface of the filling material, carrying with it atmospheric oxygen, so that the voids are continually filled with air, the oxygen content of which in practice does not become seriously exhausted. The air supply under the best conditions may amount to five times the volume of sewage. The material over which the films of sewage continuously trickle supports an active growth of microorganisms; and everything favors their maximum activity. The process is analogous to the cultivation of acetic-acid bacteria in vinegar manufacture by the flow of weak alcoholic liquor over shavings. The complications introduced by " a series of compensating errors of surfeiting and starvation " in the contact bed are exchanged for a simple and constant condition. Under the name of the trickling filter, the percolating filter, the " intermittent continuous " filter, the sprinkling filter, etc., this process has come nearer than any other filtration process to realizing the ideal conditions for rapid purification.

The first description of a method for sewage treatment based on the plan of trickling over coarse material with natural aeration was published by Stoddart in 1893. In the next year the same investigator exhibited a model at the Bristol meeting of the British Medical Association in which sewage and other liquids

were discharged in drops over a filter of coarse chalk. A solution of ammonium sulphate containing 140 parts per million of nitrogen was almost perfectly nitrified at a rate of 11,600,000 gallons per acre per day. Sewage was completely nitrified at a rate of 1,200,000 gallons per acre per day and well purified at 5,800,000 (Dibdin, 1903). A working filter was constructed by Stoddart on this principle at Horfield in 1899. The same principle was independently worked out by Corbett, the borough engineer of Salford, in a series of experiments begun in 1893 under the inspiration of the work of the Massachusetts State Board of Health. He first used wooden troughs for distribution. Later he raised these troughs to a height of several feet above the filters, so that the sewage fell on the bed in a shower. Finally he sprayed the sewage over the surface from sprinkler nozzles such as are in use to-day. Corbett's work played an important part in the development of the details of construction of the trickling bed. He showed that aeration could be improved by laying a false bottom of half-pipes on the floor of the filter; and he studied the effect of dividing the bed into horizontal layers separated by an air space.

Besides Stoddart and Corbett, three other pioneers, Ducat, Scott-Moncrieff and Whittaker, must be mentioned in any outline of the development of the trickling filter. Colonel Ducat strongly urged the importance of thorough aeration, building filters with open sides to attain that end, and maintained that the aerobic process alone was entirely competent for the treatment of crude sewage. He installed a small filter at Hendon in 1897. Scott-Moncrieff carried the process to a logical extreme in a series of experiments at Ashtead in 1898, described in Chapter IX.

The Principles of Purification in Coarse-grain Beds. Even less is known about the fundamental chemical and bacterial problems in the case of the trickling filter than in that of the contact bed. Nitrification and nitrosification play a part and are subject to the same laws which operate in intermittent filtration. The curves plotted in Fig. 88 bring out these two distinct processes in operation in the Technology experimental filters at Boston (Winslow and Phelps, 1907) and show the dependence of nitrification upon nitrosification and the dependence of nitrosification upon temperature. The filters started in October did

not form mineral nitrogen in large amounts until the warm weather of the following summer. By September, nitrite formation reached a maximum, and immediately afterwards the production of nitrates began in earnest, and, once started, proceeded

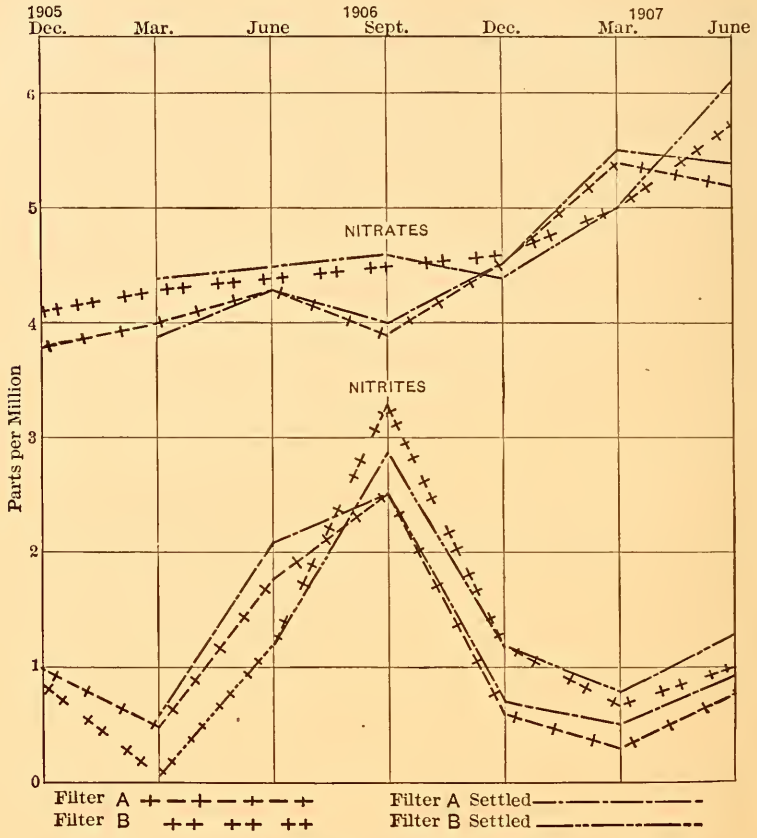

FIG. 88. Nitrification and Nitrosification in the Technology Trickling Filters.

successfully through the winter, using up the nitrites as fast as they were formed.

Sometimes, as in Scott-Moncrieff's experiments, the nitrifying processes play a predominant part. As a rule, however, the organic nitrogen is by no means entirely converted into nitrates. Frequently the total nitrogen and ammonia nitrogen values are

only reduced by 25–50 per cent; and yet the effluent may be non-putrescible, because the nitrogenous matter present is in a stable humus-like form and because the effluent contains sufficient oxygen to take care of such decomposable organic matter as may remain (see Table LXXVI).

TABLE LXXVI

RESULTS OF TRICKLING FILTRATION AT BOSTON, JANUARY-JUNE, 1907

Parts per million. (Winslow and Phelps, 1907.)

	Nitrogen as —					Putrescibility, per cent samples stable for	
	Organic nitrogen.		Ammonia N.	Nitrites.	Nitrates.	2 days.	14 days.
	Total.	Dissolved.					
Septic effluent.....	4.7	2.5	18.7	0	0	....	....
Filter effluent.....	3.4	1.4	12	0.8	5.3	95.1	82.6

There is of course no doubt that the processes which go on in the trickling filter are in many ways similar to those which take place in the contact bed. Physical forces of adhesion and adsorption retain solid and liquid constituents on the surfaces of the filling material and the bacteria of the surface films subsequently effect the ultimate purifying changes. Both chemical and physical actions differ, however, in detail. Instead of decomposition into free nitrogen, the trickling bed forms nitrates and stable organic compounds; and the action of the trickling bed upon suspended solids is notably characteristic. The contact bed destroys a considerable part of the suspended solids applied to it; the trickling bed does not. The net amount of suspended solids discharged in the effluent is just about equal to that applied. It must be understood, however, that the material which the trickling bed discharges is by no means the same organic matter which it receives. It is in much denser particles, as indicated by the rapidity with which it may be removed by sedimentation. Under the microscope the various kinds of débris characteristic of sewage are absent, and instead there are in the main structureless flocculent masses, like the amorphous matter found in the microscopical examination of water. Futhermore, the filter has, in relation to suspended solids, a peculiar and definite annual cycle. During the autumn and winter months it stores suspended solids; and in

the spring it discharges the accumulation. The history of this phenomenon in the case of the Technology filters is shown in Fig. 89 (Winslow and Phelps, 1907). The exact cause of the spring break-up in the beds is uncertain. The low vitality of bacteria in winter and the increased growth with the warm weather of spring may both play a part. In any case, the delicate

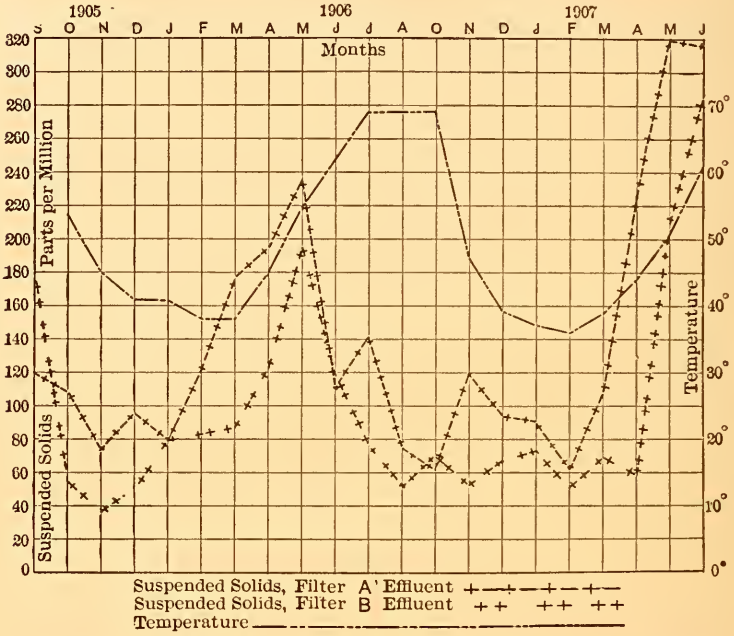
1905
1906
1907
S O N D J F M A M J J A O N D J F M A M J
Months
Parts per Million
Suspended Solids
Temperature
320 300 280 260 240 220 200 180 160 140 120 100 80 60 40 20 0
70° 60° 50° 40° 30° 20° 10° 0°
Suspended Solids, Filter A' Effluent
Suspended Solids, Filter B Effluent
Temperature

FIG. 89. Yearly Cycle of Suspended Solids, Technology Trickling Filters.

adjustment of a biological mechanism is here strikingly illustrated.

Factors which Control the Efficiency of the Trickling Bed. Rudolph Hering (1909) in a very suggestive summary of the principles of sewage treatment, has pointed out that the main engineering factors in the operation of the trickling bed are three in number — the area of bacterial surface on the stones in the filter, the amount of oxygen available for the oxidation of the organic matter and the time of exposure. The temperature and the proportion of suspended matter in the sewage are classed as minor variables.

The bacterial surface will of course vary inversely with the size of the filling material, and also, according to many authorities, with the nature of its surface. Minor irregularities will soon be covered over, but Hering believes that the deeper films in the depressions will be active, nevertheless, through their absorptive power; and he calculates, assuming the surface of a glass sphere as 1, that the effective surface for gravel of the same average diameter would be 1.5, for broken stone 2 and for slag 2.5 or over. These conclusions are scarcely borne out by experimental results. At Hamburg, Salford and York, slag was better, but only slightly better, than smoother material; at Buxton and Tipton, on the other hand, coal proved superior to more porous substances. In any case, the size of the filling material is the chief controlling factor. Hering calculates that at Hanley $\frac{3}{8}$-inch saggers yield a bacterial surface of 135 square feet in every cubic foot of the bed; at Birmingham, $1\frac{3}{4}$-inch slag and granite give 60 square feet; and at Wilmersdorf 5-inch coke gives only 25 square feet. If we put aside Hering's correction for the roughness of the material, these figures become respectively 90, 34 and 10, an even more striking comparison.

The amount of air supply required is less perfectly known than any other factor. Rideal calculates that to convert the nitrogen of sewage into nitrates one-half gallon of air must be supplied for every gallon of English sewage. Hering estimates the air supply for the three plants cited above at from 4 to 7 cubic feet per day for the sewage of each person treated.

The time of passage through trickling beds varies with the rate of flow and with the size of the filling material; and Clifford (1908), in a very pretty study of this problem, has expressed the relation by a mathematical formula. He distinguishes two portions of liquid in a trickling bed at a given time, — the absorbed water, held in the pores of the material, and the interstitial water, on its surfaces. The absorbed water was determined by weighing the dry material, then submerging it for a considerable period, wiping with a cloth and weighing again. It amounted to 2.4 United States gallons per cubic yard of bed with gravel, and to 12.9 gallons with coke breeze. The interstitial water was found by subtracting the absorbed water from the total water held in the bed at a given time, when in actual operation. Interstitial water varied from 9.5 to 19.6 United States gallons per cubic yard,

according to the fineness of the filling material, the rate of filtration being 240 United States gallons per square yard per day. An increase in rate of 60 gallons per square yard increased the interstitial water by 0.5 gallon.

The time of passage through a 2.4-foot bed was found by Clifford to vary directly with the amount of interstitial water (or, in other words, inversely with the size of the material), and inversely with the rate of application. He expressed the relation by the formula

$$cT = \frac{I}{R},$$

T being the time in minutes, I the interstitial water in gallons per cubic yard of bed, R the rate of application in gallons per square yard per hour, and c a constant. The average value deduced for c, with a bed 3 feet deep, was 0.03. The various depths of different materials necessary to secure a time of passage of 100 minutes are tabulated below:

TABLE LXXVII

DEPTH OF BED NECESSARY TO SECURE A 100-MINUTE PERIOD

(Clifford, 1908.)

Gravel.		Coal.		Slag.		Clinker.	
Grade.	Depth.	Grade.	Depth.	Grade.	Depth.	Grade.	Depth.
In.	Ft. In.	In.	Ft. In.	In.	Ft. In.	In.	Ft. In.
1 — 3/4	8 3	3/4 — 5/8	7 2	1 1/4 — 3/4	6 1	1 — 3/4	6 1
3/4 — 5/8	7 3	1/2 — 3/8	5 2	3/4 — 5/8	5 4	5/8 — 3/8	3 9
5/8 — 1/2	6 7	3/8 — 1/4	4 5	5/8 — 3/8	4 9	.....	.....
1/2 — 1/4	4 4	1/4 — 1/8	4 0	3/8 — 1/4	4 0	.....	.....
.....	.....	.....	.....	1/4 — 1/8	3 10	.....	.....

Hering has combined the three variables of bacterial surface, air supply and time in the following formula:

$$p = bat,$$

where p = the degree of purification;

b = the area of bacterial film per person in square feet;

a = air supply necessary for the purification of the sewage of one person per day in cubic feet; and

t = the time of passage through the filter in minutes.

The application of this formula is illustrated by the following table:

TABLE LXXVIII

FUNDAMENTALS OF SEWAGE PURIFICATION

(Hering, 1909.)

City.	*b* Bacterial surface per person, sq. ft.	*a* Air supply per person, cu. ft. per day.	*t* Time of purification, minutes.	*p* Calculated degree of purification.	Actual character of effluent.
Hanley.....	710	4	76	216,000	Excellent
Birmingham.....	367	5	40	73,400	Fair
Wilmersdorf.....	160	7	15	16,800	Tolerable

The chief practical variables in the construction of trickling beds are the size of the material and the depth, the former affecting Hering's factor, *b*, and both affecting *t*. Perhaps some day accumulated experience will make it possible to fix a value for *p* by which we can determine for any given sewage the depth of a bed which will yield a stable effluent with a given material, or conversely the largest material which will suffice for purification with a given depth.

Construction of Trickling Beds. Reduced to its simplest terms, a trickling filter is a heap of selected filtering material; and some of the earlier filters, at Bristol, for example, were little more than this in actual fact (see Fig. 90). Sometimes local conditions of head make it necessary to build the beds below the surface of the ground; in this case it has been thought economical to form the sides of the excavation to their natural angle of repose. In some of these cases the filter proper is surrounded by a battered wall of filtering material. This, however, is not an economical method. At Birmingham it was found that building a rubble wall around an acre filter cost only half as much as the extra labor and material required for a battered wall of filtering material. As a general rule, therefore, it will be found best to build trickling filters with structural walls of some sort. The floor of the filter must in any case be smooth and impervious, so as to insure prompt and efficient drainage; cement concrete is generally the most suitable material.

The general form of the individual beds, whether circular, rectangular, triangular or hexagonal, as well as their relation to each

other, will be largely determined by the form of distributing device adopted. Movable distributors require circular or rectangular beds or groups of hexagons; fixed sprinklers allow the filters to be laid out in large continuous areas of any indicated form.

Fig. 90. View of Trickling Filter at Bristol (courtesy of F. Wallis Stoddart).

Underdrainage and Ventilation. Careful underdrainage is one of the most important essentials in the trickling filter, not only for the removal of the liquid with its load of suspended solids, but also for the satisfactory aeration of the bed. Sometimes branch underdrains are laid at intervals; it is perhaps better to construct what is practically a false floor by the use of some of the devices shown in Fig. 91. The underdrains used at Columbus, Ohio, are of 6-inch half-tile laid in the manner indicated in Fig. 113. Main collectors should be of ample dimensions, and if possible so exposed that the wind may have free access to them. Those designed for the Columbus plant are concrete drains 24

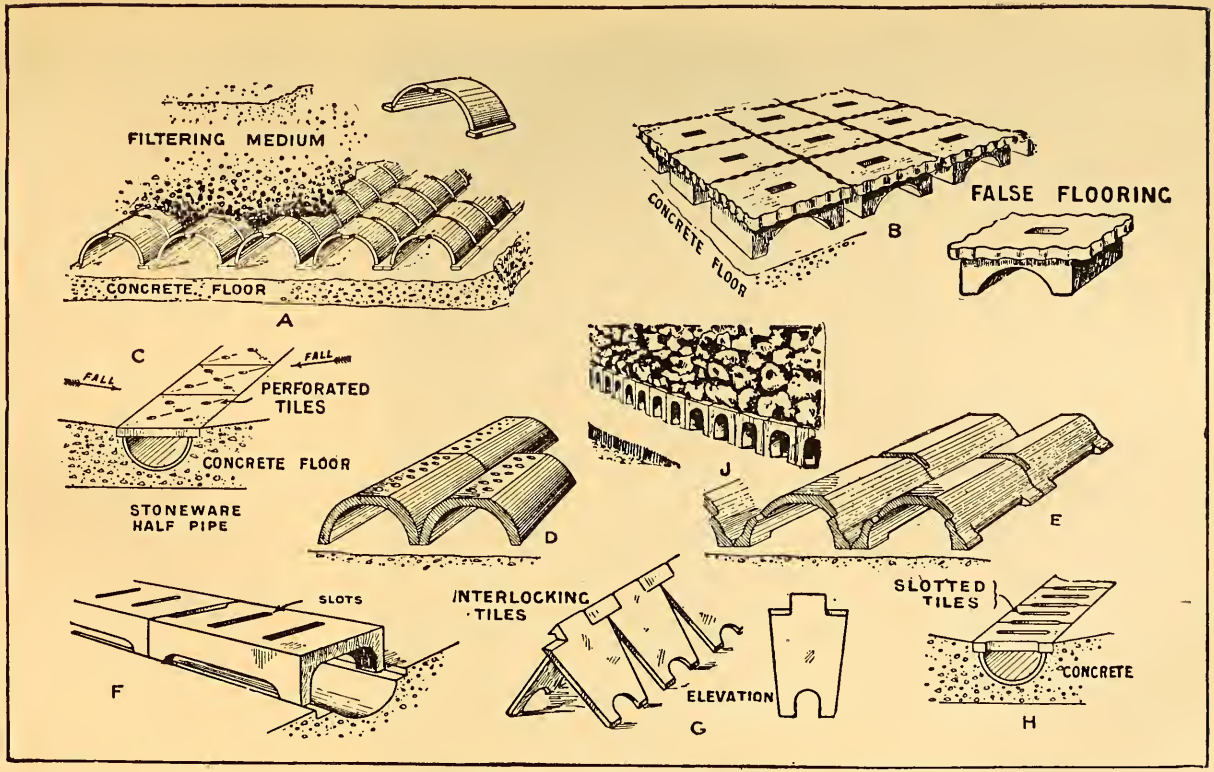

FIG. 91. Types of Underdrains for Trickling Beds (copied by permission from *Sewage Disposal Works* by H. P. Raikes, Constable Co., London).

and 36 inches in diameter. It is well to provide that underdrains should converge toward the center of the filter for economy of head and where possible they should pass out through the walls at their upper ends to allow for flushing.

At Fitchburg, Mass., Metcalf and Eddy laid out the filter floors with shallow drains across which were placed narrow cement beams (17 inches x 4 inches x $1\frac{1}{8}$ inches) spaced $1\frac{1}{8}$ inches apart, with cobbles above to support the filtering material.

Special facilities for aeration are provided in many English

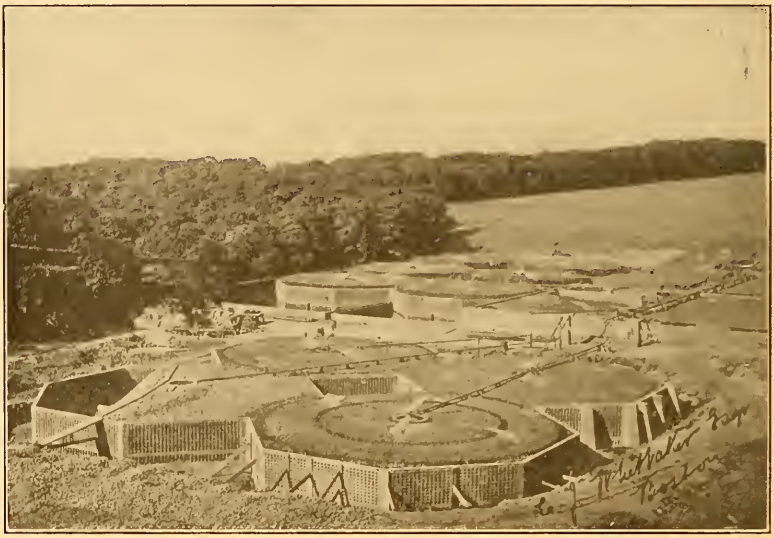

Fig 92. Whittaker-Bryant Filter at Accrington (courtesy of Mr. Whittaker).

filters by open construction of the walls. The walls of the Ducat filter were built of open drainpipe inclined upward and connected with aerating drains at intervals in the body of the bed. The Whittaker-Bryant filters at Accrington and elsewhere are octagonal in shape, with walls of open brick and central open brick work aerating wells (Fig. 92).

The trickling filters at Atlanta have been provided with 8-inch cast-iron pipe risers (72 per acre) connected below with the underdrains and terminating above the filter material in revolving cowls. The necessity for such provision is somewhat

doubtful if free underdrainage is provided. In the treatment of fresh sewage, as at Reading, it has been found that the sewage as it enters the filter after discharge from sprinkler nozzles is 60–80 per cent saturated with oxygen. Rudolph Hering (1909) however believes that the open-side filters in England and the special air pipes at Wilmersdorf have proved of distinct advantage, and holds that the resting of the filters which has been found necessary at Birmingham might have been avoided by similar provisions. A considerable amount of air must certainly be supplied to the interior of the filter from some source. Fowler has calculated that the sewage going on at the top even when completely saturated with air, contains only one-fifteenth of the oxygen necessary for purification.

Filling Material. As in the case of contact beds, evidence in regard to the relative value of various kinds of filling material is somewhat conflicting. At Salford, slag was found somewhat better than polarite, gravel, coke, or clay. At York a well-controlled series of investigations indicated, as shown in the table below, that coke and boiler slag (clinker) are slightly better than brick and blast-furnace slag.

TABLE LXXIX

EFFICIENCY OF TRICKLING FILTERS OF DIFFERENT MATERIALS AT YORK, ENGLAND

Parts per million. (Bredtschneider and Thumm, 1904.)

	Nitrogen as —		Oxygen consumed in 4 hours at 80° F.
	Albuminoid nitrogen.	Nitrates.	
Raw sewage	13.9	0	82.9
Broken-brick effluent	1.4	18.4	10
Blast-furnace slag effluent	1.2	18.8	9.6
Coke effluent	0.9	23	7.1
Boiler-slag (clinker) effluent	1	22	6.9

At Buxton, effluents from destructor breeze and coke showed 0.8 and 0.9 part of albuminoid nitrogen against 0.4 part for that of a coal filter and 0.8 and 0.7 part of nitrates against 3.4 parts (Barwise, 1904). Calmette (1909) has reported excellent results from experimental filters filled with fragments of peat; but he found that cinders mixed with carbonate of lime gave the best results of all. At Philadelphia (1911) direct examination

showed that both bacterial jelly and suspended solids adhered much more readily to rough slag than to smooth gravel.

On the whole, it seems probable that any hard material will serve well for the trickling filter. Raikes (1908), in reviewing English experience with various materials, mentions particularly burnt clay, coal, coke, gravel, slag, sandstone, granite, furnace

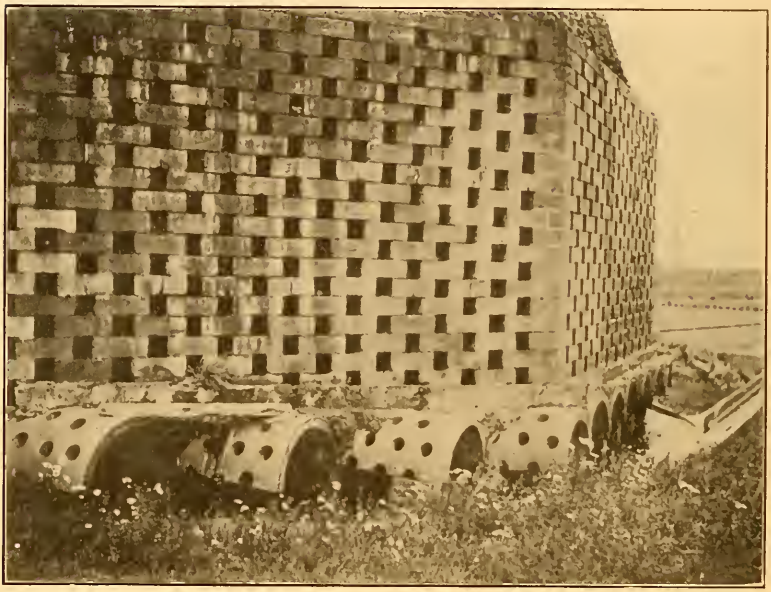

Fig. 93. Open Brickwork Construction of Trickling Filter at Leeds.

clinker, overburnt bricks and saggers. Burnt clay has worked badly, soon disintegrating into powder. Coal may serve, if a hard variety can be cheaply obtained, but it is more liable to break down than harder materials. Coke settles considerably and forms a good deal of wasteful dust when put in place. Sandstone takes a long time to drain and forms much dust when it is broken. Raikes holds that gravel is too smooth for supporting the most luxuriant bacterial films. Slag and furnace clinker are satisfactory if of good quality, but the quality varies greatly, depending upon the source of the material. Some clinkers are soft and friable, and slag containing lime or sulphur is apt to disintegrate. Overburnt brick, saggers and granite are hard and impervious and form excellent filling material where they are available.

The size of the filtering material to be used will be largely controlled by the character of the sewage treated, and particularly by the amount of suspended solids it contains. Considerable latitude may be allowed in this regard without seriously affecting purification.

Hering's calculations, cited on page 325, show that the Wilmersdorf beds of 5-inch coke with a bacterial surface of only 160 square feet per person give a tolerable effluent, while ordinary filters, like those at Birmingham, of $1\frac{3}{4}$ inch slag have twice as large a surface, and the fine beds at Hanley ($\frac{3}{8}$-inch saggers) have over four times as much. Barwise (1904) suggests $\frac{1}{8}$- to $\frac{1}{2}$-inch material, and Raikes (1908) places the limits at $\frac{1}{4}$- to $\frac{3}{4}$-inch. Among the witnesses before the Royal Sewage Commission, Garfield recommended $\frac{1}{16}$ to $\frac{3}{16}$ inch, Ducat $\frac{1}{8}$ to $\frac{1}{2}$ inch, Corbett $\frac{3}{16}$ to $\frac{3}{4}$ inch, Candy $\frac{3}{16}$ to $\frac{1}{2}$ inch for fine and $\frac{3}{4}$ inch to 3 inches for coarse beds, Harding $\frac{1}{4}$ inch to $1\frac{1}{2}$ inches for fine and over 3 inches for coarse beds, Whittaker 1 inch to $1\frac{1}{2}$ inches, and Stoddart 2 to 3 inches.

The efficiency of a trickling filter of fine material is well illustrated by the Cleveland studies of a 4-foot bed of $\frac{3}{4}$-inch slag. This filter treated Imhoff tank effluent at a net rate of 1,600,000 gallons per acre per day and gave the average results indicated below.

TABLE LXXX

EFFICIENCY OF TRICKLING FILTER OF $\frac{3}{4}$-INCH SLAG

June-Nov., 1913. Cleveland (1914.)

	Nitrogen.			Oxygen consumed.	Total suspended solids.	Dissolved oxygen, per cent saturation.	Relative stability.	Bacteria per c.c. at 37°.
	Total organic.	Ammonia.	Nitrates.					
Composition of effluent parts per million..	2.7	5	6.8	21	30	75	95	63,000
Per cent purification.....	58	65	....	44	78	.....	....	....

The degree of purification is very high, particularly as regards suspended solids. The solids in the effluent were however rising through the course of the experiment and if the study had been prolonged would no doubt have continued to do so. At no time did the dissolved oxygen in the effluent fall below 54 per cent of saturation.

In the United States, with weaker sewage and less complete preliminary removal of solids than are common in England, coarser beds are generally recommended. At Reading 1½- to 4-inch blast furnace slag was used; and at Columbus 1¼- to 4-inch limestone. At Boston the Technology experiments indicated that crude sewage can be treated successfully in beds of 1½-inch gravel, while a ½-inch bed clogged and pooled badly.

The material in filters built more recently in this country (Akron, Atlanta, Baltimore, Fitchburg and Gloversville) has generally varied between 1-inch and 2½-inches diameter.

The depth of the trickling bed is controlled, on the one hand, by the strength of the sewage to be treated, and on the other hand by the size of the filtering material. With material between ⅛ and ¼ of an inch, the sewage travels downward at a rate of from ½ inch to 1 inch per minute, while with ½- to ¾-inch material this rate is nearly doubled. With quite fine material a shallow filter will serve, and there may be no particular gain in deepening it. Thus, Table LXXXI from the Hanley experiments, shows that a bed of ⅛-inch to ¼-inch material carefully dosed by a moving distributor effected practically complete purification in a depth of only two feet, and excellent results even in one foot:

TABLE LXXXI

PURIFICATION IN TRICKLING BEDS OF VARIOUS DEPTHS AT HANLEY

Parts per million. (Raikes, 1908.)

	Suspended solids.		Ammonia N.	Albuminoid N.	Oxygen consumed, 3 min.	Nitric nitrogen.
	Total.	Volatile.				
Sewage.....	635	285	21.54	9.72	18.62	0.2
Detritus tank..	170	68	16.43	4.86	9.75	0.2
Septic tank....	76	38	17.16	3.40	8.36	0
Filter, 1 foot...	2.5	1.6	0.36	0.52	0.93	6.4
Filter, 2 feet...	0.9	0.5	0.20	0.37	0.77	18.2
Filter, 3 feet...	1.4	0.6	0.09	0.31	0.60	17.5
Filter, 4.5 feet.	.....	.....	0.43	0.27	0.70	17

Practically, four feet should probably be fixed as a minimum, even for a fine bed, to avoid the danger that streams of unpurified sewage may pass through channels formed by irregular packing of the material. With coarser filling the depth may have to be increased in proportion to the coarseness of the material, the

strength of the sewage and the rate of filtration. Results obtained at Leeds with three successive filters of coarse coke made it clear that $3\frac{1}{2}$ feet was insufficient for adequate purification.

TABLE LXXXII

EFFICIENCY OF TRICKLING FILTERS AT LEEDS, ENGLAND

Parts per million. (Dibdin, 1903.)

	Depth, feet.	Total solids.	Suspended solids.	Nitrogen as —		Oxygen consumed in 4 hours at 80° F.
				Ammonia nitrogen.	Albuminoid nitrogen.	
Sewage.....	....	1760	631	27.6	12.2	127
Effluent No. 1....	3.5	1250	275	18.5	7	62.5
Effluent No. 2....	6	1060	113	13.3	5.0	39.6
Effluent No. 3.....	8.5	1010	110	9.7	3.5	27.6

An interesting study of the effect of depth of beds on trickling filter efficiency was made at Milwaukee (1915). Two beds of 1-inch to $2\frac{1}{2}$-inch crushed limestone were compared, each treating Imhoff effluent. The difference in the results obtained is indicated in the table below.

TABLE LXXXIII

COMPARATIVE EFFICIENCY OF TRICKLING BEDS OF DIFFERENT DEPTH

Parts per million. (Milwaukee, 1915.)

	Suspended solids.	Nitrogen, total organic.	Ammonia N.	Albuminoid N.	Nitrate.	Oxygen consumed.	Dissolved oxygen.	Bacteria per c.c. 20°.
8-foot bed..	96	23.6	14.7	4.8	5.7	55	7.1	752,000
6-foot bed..	121	26.9	15	5.4	2.6	67	6.3	1,003,000

At Reading and Columbus, with $1\frac{1}{4}$- to 4-inch material, the filters have been designed 5 and $5\frac{1}{2}$ feet deep respectively. At Wilmersdorf, with 5-inch coke filling, even 8-foot beds do not yield a more than tolerable effluent. If filters deeper than four feet are undesirable on account of insufficient head, the same end may be attained by increasing the area and decreasing the rate of filtration. The British Royal Commission (R. S. C., 1908) concluded that a given quantity of coarse material would effect approximately the same degree of purification whether arranged

in a deep or in a shallow bed. With fine material, however, they point out that the best results can be obtained for a given amount of material with shallow beds.

American engineers are not entirely in agreement as to the most efficient depth for trickling filters. Baltimore studies favored rather shallow beds and George W. Fuller has been inclined to maintain the same position. Metcalf and Eddy (1916) on the other hand have prepared a table of results obtained at Gloversville, Lawrence and Worcester expressed in terms of applied nitrogen in grams per cubic yard of filtering material, which seem to indicate that deep beds are most effective. They also believe that the cost of deep beds is materially less for a given cubic yardage. Results tabulated below, obtained with beds of different depths operated so as to give an effluent of approximately the same character, speak strongly in favor of deep beds.

TABLE LXXXIV

EFFICIENCY OF TRICKLING BEDS OF DIFFERENT DEPTHS

(Clark, 1915.)

Depth of bed.	Sewage treated, gallons per cubic yard.	Parts per million.				Percentage of stability.
		Nitrogen as —			Oxygen consumed.	
		Ammonia N.	Albuminoid N.	Nitrates.		
Ft.						
4	51.7	18.6	3	16.3	18.9	85
6	60.6	12.9	2.9	18.1	17.9	91
8	139.8	13.6	3.7	16.6	21.7	88
10	231.8	17.3	3.9	17.6	23.8	85

The Distribution of Sewage on Trickling Beds. The distribution of the sewage in a fine spray over its surface is the chief problem in the successful operation of a trickling bed. The attempts of Lowcock and Waring to secure distribution by spreading a layer of fine material over the whole surface of the bed have been already described. This method is very rarely attempted, however, at the present day. Scott-Moncrieff and Ducat originally used tipping buckets and troughs placed at intervals over the filter, relying on the dash to distribute over intermediate areas. This plan has been tried experimentally at Hendon and Leeds. The distribution is of course imperfect, so that deep beds of rather coarse material should be used to

insure purification without serious ponding. The apparatus requires pretty constant supervision to keep it in order; but the method has the advantage of adjusting itself easily to variations of flow and for small plants may prove satisfactory.

A third distinct type of distributor is the dripping tray, devised by F. Wallis Stoddart for use at Horfield. It is practically a series of channels, over the sides of which the sewage overflows continuously, dripping from a series of points on the underside, 360 points being allowed to a square yard. Theoretically it should secure a very even distribution. The channels are liable to buckle, however, and it is difficult to keep them level. Furthermore, they are subject to serious clogging from fungous growths (Barwise, 1904).

Neither of these methods of distribution has been widely used. In England two systems have attained rather general acceptance, one involving the use of movable sprinklers which rotate or travel back and forth over the surface of the bed, driven by power or by the head of the sewage, and the other depending on the spraying of the sewage in jets from the orifices of a fixed system of piping. The first type, including all the various forms of movable distributors, is still in most general use in England.

Movable distributors may again be divided into two classes, according as they are driven by the head of the sewage itself or actuated by special applications of power. The first to come into use were of the former type, and were generally operated on the principle of the Barker's Mill. They consisted of a number of perforated iron pipes radiating from a central pillar and revolving about it, receiving sewage from the center and discharging through holes all on one side of the pipes, so that the reaction of the escaping jets caused the arms to revolve in the opposite direction. A view of the Candy-Whittaker distributor in Fig. 94 shows the general appearance of this form. In small plants the sewage is supplied to the arms from overhead by an annular trough revolving on a ring of ball bearings around the central supply pipe. With larger filters and longer arms, requiring a greater head of sewage, the construction of these troughs is difficult; and in any case stoppages in the revolving arms would lead to serious overflows at the center. In most cases, therefore, the arms are fed from below. This means that the apparatus must be water-tight under pressure; and this end is difficult to

attain without introducing so much friction as to waste a considerable proportion of the available head.

Another form of revolving sprinkler designed by Mather and Platt is driven by a turbine wheel in the central pillar. Still a third entirely different type of automatic distributor depends on power developed by discharging the sewage over a sort of movable

FIG. 94. Trickling Beds Equipped with Candy-Whittaker Distributor (copied by permission from Barwise, 1904).

water wheel so that the impact of the sewage from the trough revolves the wheel. A distributor of this general plan (the Fiddian) is shown in Fig. 95, as adapted to a rectangular bed. The feed pipe is a siphon which moves in a trough at the side of the bed. Alternate sections of the discharge buckets empty in opposite directions, and at the end of each excursion a lever collides with a buffer and deflects the sewage to the other set of buckets, which drive the apparatus back again to the end of the bed from which it started. These Fiddian distributors are costly and require more power to drive them than do the revolving pipes. Their freedom from small openings liable to clogging, their adaption to rectangular beds and the fact that they will operate under widely varying heads, are strong points in their favor.

Where there is insufficient head for any of these automatic devices, power-driven sprinklers have been introduced in many English towns. One of the earliest of these was devised by Mr. Scott-Moncrieff and is shown in Fig. 96 as it was used in experi-

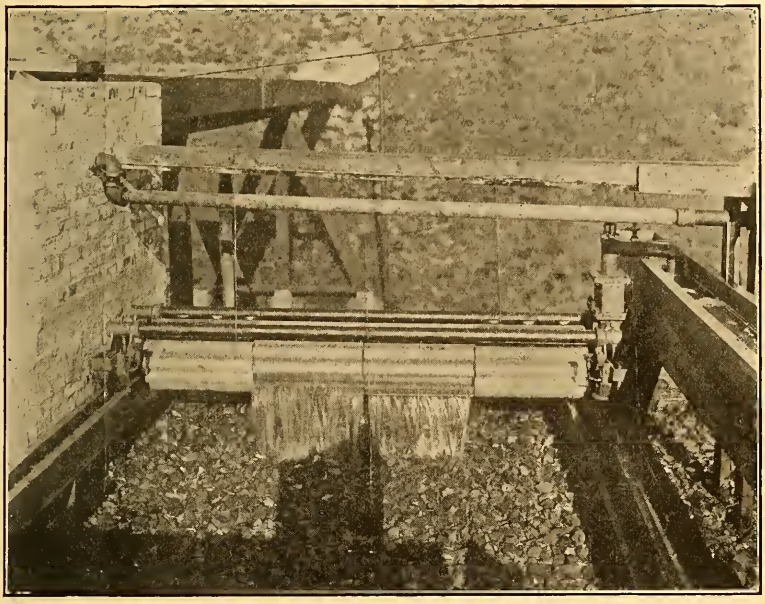

FIG. 95. Fiddian Distributor for Rectangular Beds, Technology Experiment Station, Boston.

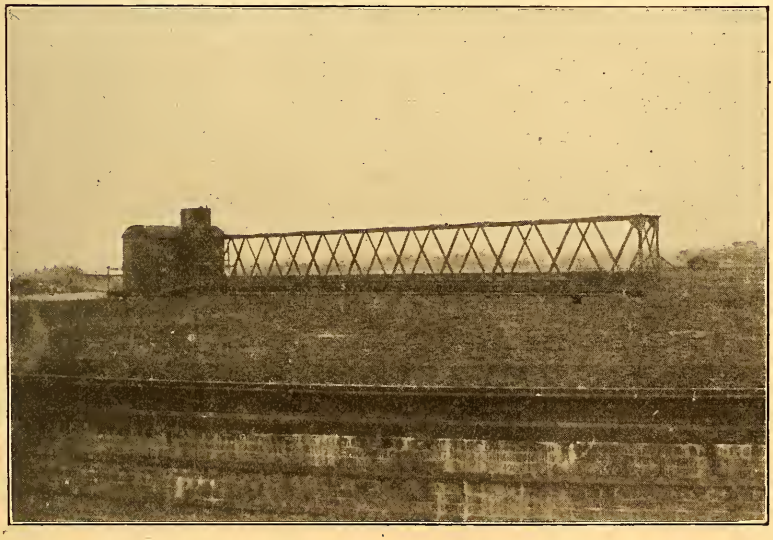

FIG. 96. Scott-Moncrieff's Revolving Distributor at Birmingham.

ments at Birmingham. It consists of a large open trough revolving about a vertical supply pipe at its center and supported on a circular rail at the periphery. The sewage overflows from a small side trough divided into sections, to which it is admitted from the main trough in amounts proportional to the circular path covered by each section. The propelling power is furnished by an oil engine carried on the outer end of the trough. The distribution effected is excellent, but the weight of the apparatus is great and the cost of operation very high. At Hanley the distributor for a quarter-acre bed weighed 12 tons; a 45-lb. bridge rail around the filter was worn out in two years and a half; and the cost of renewal and supervision amounted to ten dollars per million gallons of sewage distributed. In the method finally adopted at Hanley, the distributor itself is a perforated iron pipe moved back and forth on wheels by means of a wire rope, driven from an electric motor provided with automatic gear by which the motion is reversed at the end of each excursion.

Traveling distributors were installed for the first time in this country (on a practical scale) at Springfield, Mo., in 1913 (Potter, 1916). The plant in question was designed for a capacity of 3,500,000 gallons per day and the mechanical distributor was selected in order to economize head. The apparatus was designed to distribute the sewage with a loss of head not to exceed 12 inches when applied at a maximum rate of 720 gallons per square yard per day. Each distributor was supported on three rails spaced 25 feet apart with a length of travel of 200 feet. Each pair of the six distributors was operated by an endless wire cable, three sets of cables being driven by a 6-hp. Otto gasoline engine which gave them a velocity of 38 feet per minute. The change of direction was accomplished by a reversing lever. After two years' experience Alexander Potter, the designing engineer, reported that these mechanical distributors had operated very successfully in spite of severe winter weather although constant attention was required. The cable drive, however, proved very expensive and the cables have been replaced by a direct motor drive, a $\frac{1}{4}$-hp. motor being mounted directly upon the frame of the distributor. First cost, power cost and particularly maintenance cost have all been materially diminished by this change, the total cost of distribution (including fixed charges, operation and maintenance) having been

reduced from $0.92 to $0.28 per million gallons of sewage treated.

Most of these movable distributors give excellent results, so far as evenness of distribution is concerned, They are costly, however, to install; they are liable to be frequently out of order under the most favorable conditions; and in spite of the Springfield experience we may doubt whether they could generally be used successfully in a severe winter climate without protecting the whole filtering area by a roof. For these reasons they have never found favor in the United States; and there appear signs of reaction against them, even in England. At Birmingham Mr. Watson (1907) reports that the cost of sprinkler nozzles was only $2500 per acre against $5000 to $20,000 for movable distributors; and that the movable distributors were out of order for 8–28 per cent of the time.

Distribution from Fixed Sprinklers. The method of dosing trickling beds from fixed sprinkler nozzles was first, perhaps, developed at Salford, after various other attempts, with troughs and with a thin layer of sand laid over the surface of the main filter. Disc-like caps were placed over the openings of the pipes in some early experiments, in order to secure a good spray. Then the attempt was made to get a spray by the impact of two converging flows, and finally a special form of opening was designed to give a rotating movement to the stream. Two of the Salford nozzles are shown in Fig. 97. At Chesterfield and other towns in Derbyshire metal plates were placed over the orifice to break the sewage into fine spray. Most of the later types of nozzles attain this end by a cone or plug placed directly over the orifice and supported either by a central rod or by lateral arms. The Birmingham nozzle, shown in Fig. 97, has a $\frac{3}{8}$-inch opening through which passes a $\frac{7}{32}$-inch shank supporting a plug, which breaks the sewage up very effectually. Nozzles with such small openings clog badly, and at Birmingham it requires the constant attention of one man to every one and a half acres to keep them clear. In the United States larger orifices are therefore preferred, like the Columbus sprinkler shown in Fig. 97, which has a clear aperture of $\frac{9}{16}$ of an inch with an inverted cone supported above by lateral arms. The general character of the spray formed by these sharply contrasted sprinklers is indicated

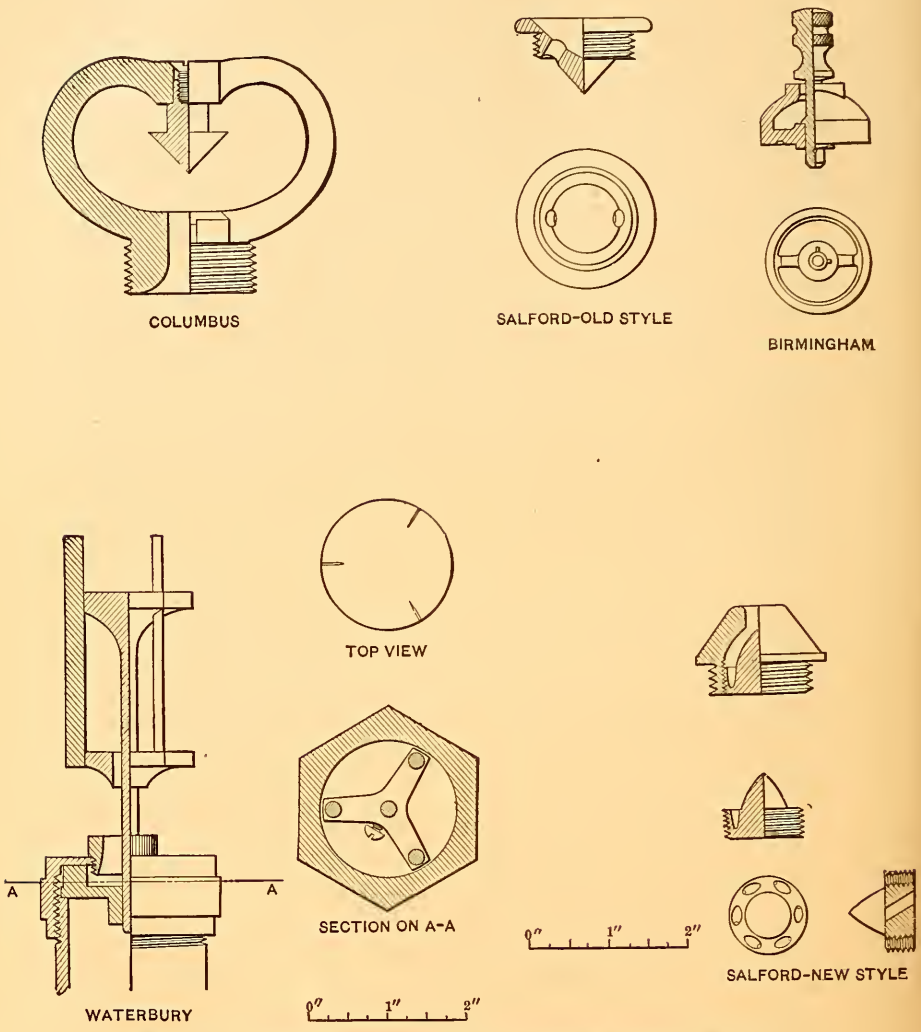

FIG. 97. Types of Nozzles for Fixed Sewage Sprinklers (Winslow, Phelps, Story and McRae, 1907).

in Figs. 99 and 100; and the Columbus plant in actual operation is shown in Fig. 114.

An excellent type of nozzle designed at Worcester, Mass., is shown in Fig. 98. Its principal features are a large dome with a minimum of interior obstruction and a circular orifice with the distributing cone supported on a spindle locked in to a lateral bar by a pin. The cone has a rim projecting from its upper base which aids in spreading the spray.

The spacing of the orifices and the head under which they must be operated naturally vary widely, according to the type of nozzle used. Taylor (1909*c*) and others have made detailed studies of distribution from fixed sprinkler nozzles, including as independent variables the cone angle, orifice pressure, orifice elevation, blade length and nozzle pressure.

An entirely different type of fixed sprinkler devised at the Sewage Experiment Station of the Massachusetts Institute of Technology has distinct advantages for certain installations. Instead of spraying upward from closed pipes, the sewage in this system of gravity distribution drops from holes in the bottom of open troughs on to concave discs, or dash plates, from which it splashes upward and outward. The spray from one of these gravity distributors is shown in Fig. 101. Careful studies have indicated that a 3-inch disc with a curvature corresponding to a radius of 2 inches gives the best results. A total head of 4 feet between trough and filter is satisfactory; but the best distribution is attained with a total head of 6 feet, the disc itself being raised 2 feet above the surface of the bed (Winslow, Phelps, Story and McRae, 1907). The discs can best be placed about 11 feet apart, each taking a discharge of 4 gallons a minute for a rate of 2 million gallons per acre per day. This combination gives a distribution intermediate between that obtained with the small Birmingham nozzle and the large Columbus nozzle; but the smallest opening liable to clogging is larger than that of the Columbus sprinkler, and closed pipes beneath the surface of the bed are avoided. This device has been used with success at St. Anthony Park, Minnesota (Bass, 1908), and more recently at the 2,800,000 gallon plant at Mt. Vernon, N. Y. (Robinson, 1909). The Mt. Vernon studies have shown that discs with the lips slightly turned over effect a much better distribution (Hammond, 1910).

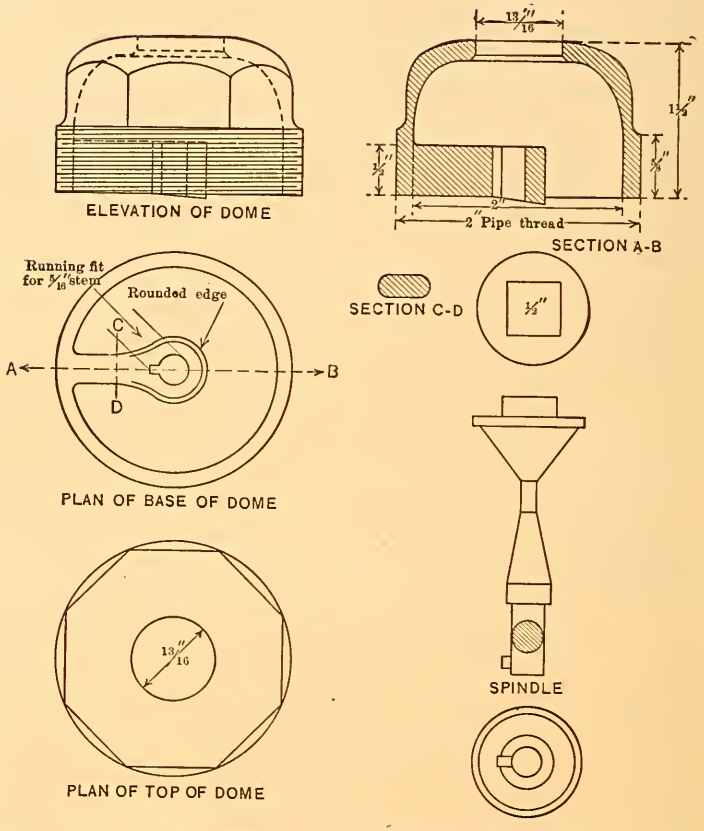

FIG. 98. Details of the Worcester Nozzle (courtesy of Matthew Gault).

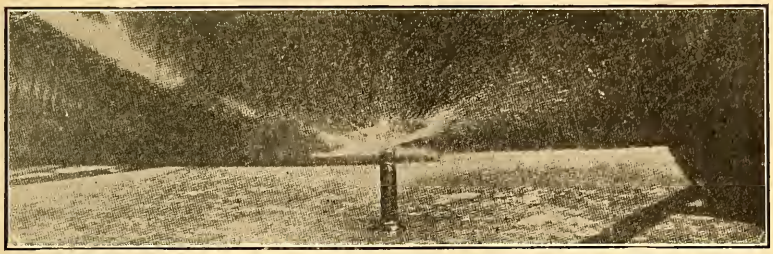

FIG. 99. Form of Spray from Birmingham Nozzle.

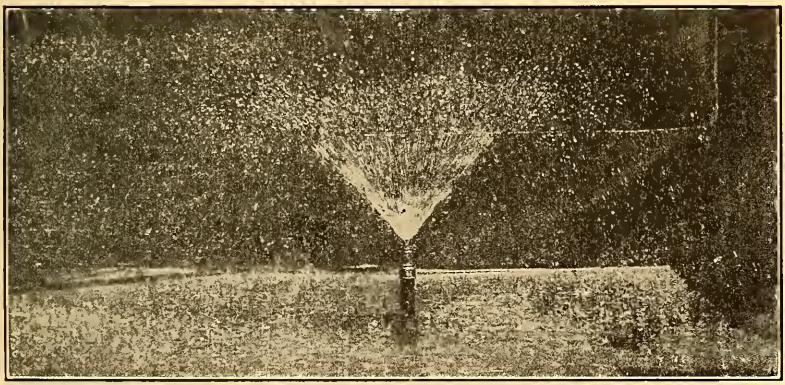

FIG. 100. Form of Spray from Columbus Nozzle.

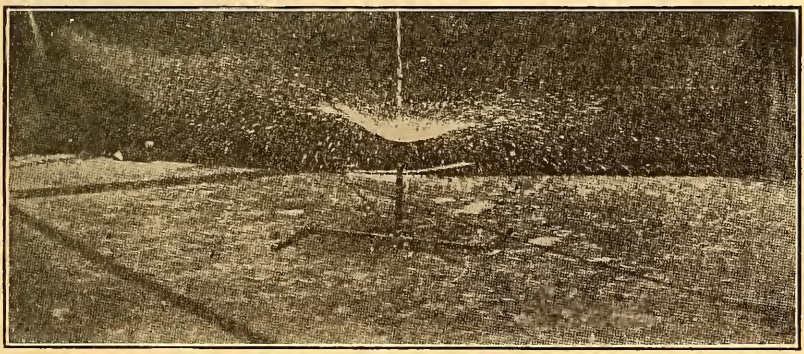

FIG. 101. Form of Spray from Technology Gravity Distributor.

Forms of Spray from Sewage Nozzles (Winslow, Phelps, Story and McRae, 1907).

Either the fixed sprinkler nozzle or the fixed gravity distributor will work well under fairly severe winter conditions, as shown by experience at the Columbus testing station, at the Technology experiment station and at the experimental plant at Worcester, Mass. Sprinkler nozzles at Worcester are shown in operation in Fig. 102, when the temperature was − 10° F. Deposits of ice and snow form between the heavily wetted circles which surround the individual sprinklers; but within each circle an ample area is kept open by the warm sewage spray. In very severe climates, as in

Fig. 102. View of Worcester Experimental Filter in Winter.

Canada, for example, the covering of trickling beds may prove necessary; and a covered plant for the Bordeaux prison near Montreal was designed in this belief.

Unquestionably all forms of fixed sprinklers are inferior in point of evenness of distribution to good movable distributors of the English type. It is a matter of considerable importance to determine how serious the inequalities may be with a sprinkler of a given type; and elaborate studies along this line have been carried out at the Technology experiment station in Boston (Winslow, Phelps, Story and McRae, 1907), at the Massachusetts State Board of Health Experiment Station at Lawrence

(Gage, 1908), at Waterbury, Conn., by W. Gavin Taylor (1909*c*) and at Fitchburg, Mass., by Metcalf and Eddy (1916). The testing apparatus used at the Technology station consisted of a circular concrete drainage trough with a pipe for pressure nozzles at the center and a trough for operating gravity distributors above. A galvanized iron box in the form of a sector of a circle revolved about the center and caught the discharge along different radii in a series of concentric 6-inch compartments. In the Lawrence experiments the spray was caught in a radial row of bottles, each bottle holding a 6-inch funnel, so spaced that the rims of the funnels were tangent to each other. The mechanical part of the tests is in any case very simple; but their interpretation is more difficult, because so many independent factors are involved. The absolute discharge in relation to the total wetted area must be considered, as well as the evenness of distribution within the wetted area, because with some nozzles the total discharge is far in excess of the amount which could be handled by the wetted area alone. The Technology and Lawrence workers have attempted to combine these various factors in arbitrary formulæ, neither of which can perhaps meet with general acceptance. It is probably best to confine the comparison of different sprinklers to the question of evenness of distribution within the wetted area, leaving the questions of head and rate and spacing of sprinklers to be separately considered. The evenness of distribution within the wetted area has been expressed by Phelps (1906) in a simple formula, as follows: The quantity, q, of liquid in each radial compartment (or in each bottle arranged along a radius) is multiplied by the distance, d, from the center, so as to give the total volume of liquid, v, discharged on the ring which the compartment represents. These quantities are then added to give the total volume, V, discharged on the whole circle. From this are derived the values, v', which would have been discharged on each ring if distribution had been perfect. From all the v' values in excess of the v values, the latter are subtracted, giving values of e, or excessive discharges. These added together give E, or the total excessive discharge, and Phelps takes for his coefficient unity minus the quotient of the excessive discharge divided by the total discharge.

$$C = 1 - \frac{E}{V}.$$

When the distribution is good, E is small and the coefficient approaches unity. A further correction must be made in practice, however, for the unwetted area between the circles. If the sprinklers are placed at the centers of squares, this means a waste of 22 per cent of the total filter area; but if they are arranged at the centers of hexagons, the waste area is reduced to 10 per cent.

The general results of the Technology tests are given in the table below. The gravity distributor was of the type described on page 341, operated under a 6-foot head. The pressure nozzles are the ones shown in Fig. 97. All were operated at a 6-foot head; but the Columbus nozzles gave slightly better results, under a 4-foot head (0.65).

TABLE LXXXV

COMPARATIVE EFFICIENCY OF FIXED DISTRIBUTORS

(Winslow, Phelps, Story and McRae, 1907.)

Type.	Rate (gallons per minute).	Sprinklers per acre, (2,000,000 gallons a day).	Coefficient.
Best gravity distributor	4.1	341	0.76
Old Salford nozzle	2.9	483	0.44
New Salford nozzle	2.1	667	0.78
Birmingham nozzle	2	700	0.84
Columbus nozzle	14.8	94	0.61
Waterbury nozzle	10.4	134	0.73

The Birmingham sprinkler nozzle with its small free opening (a $\frac{5}{64}$-inch ring) gives the most satisfactory distribution, but a large number of nozzles are required for a given area and the clogging of the small orifice calls for constant supervision. The gravity distributor gives a fairly even spray, and the Columbus nozzle, with its large orifice, naturally shows the poorest results. The serious unevenness of distribution with fixed sprinklers under constant heads is shown in Fig. 103, plotted by Taylor (1909*c*).

Circular spray nozzles should be arranged alternately in adjoining rows to reduce the dead space between and if spaced so that the discharges overlap, the undosed area may be entirely eliminated except at the edges of the bed.

The Dosing of Trickling Filters under Varying Head. Attempts have been made in two directions to improve the results obtained from sprinkler nozzles, by intermittent dosing and by

designing sprinklers to discharge a square rather than a circular spray. At Chesterfield in England the nozzles were arranged to discharge intermittently, so as to spread the sewage more evenly over a wider area. At Columbus intermittent dosing was

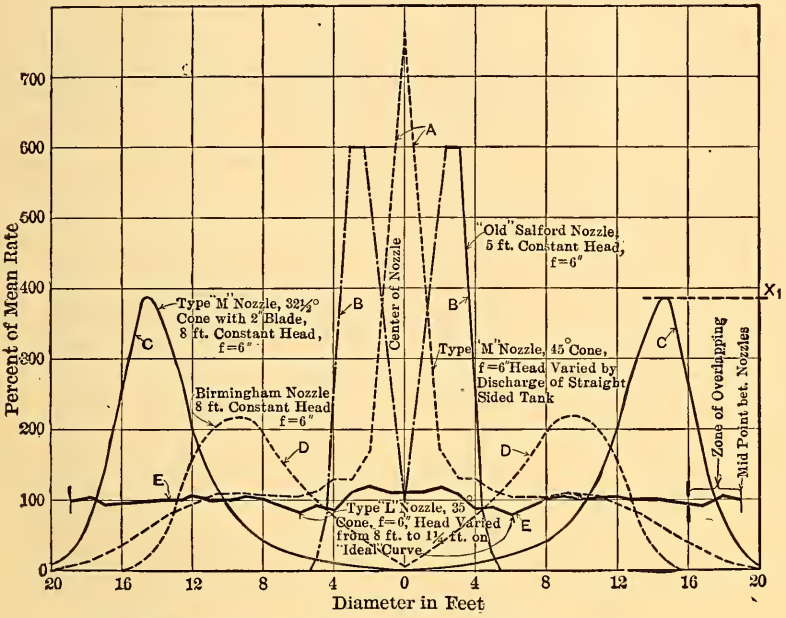

FIG. 103. Diagram Showing the Unevenness of Distribution from Fixed Sprinkler Nozzles (Taylor, 1909*c*).

made a part of the design for two reasons. In the first place, the Columbus nozzle has so large an opening that the discharge under a constant head is far in excess of 2 million gallons per acre per day on the wetted area; intermittency corrects this difficulty, at the same time that it spreads the ring of discharge in and out, and thus improves the evenness of distribution. The latter end is not, however, always attained by simple intermittent operation. In the Technology experiments it appeared that when the Columbus nozzle was dosed from a siphon tank with a head varying between 4.3 feet and 2.5 feet the only effect was to lessen the size of the ring of excessive discharge. The coefficient with a constant head of 4.3 feet was 0.65, and with varying head it ranged from 0.45 to 0.56. On the Columbus filters themselves it has

finally been found best to discharge sewage on each bed in rotation under three different heads — 4 feet, 7 feet and 9 feet (Fuller, 1909). The net result of this process is to secure excellent distribution, as shown in the table below. The coefficient calculated from the last column of figures would be 0.92.

TABLE LXXXVI

DISTRIBUTION OF SEWAGE OVER THE COLUMBUS FILTERS IN GALLONS PER SQUARE FOOT PER MINUTE

(Fuller, 1909.)

Distance from nozzles, feet.	Nozzle head.			Average for cycle.
	4 feet.	7 feet.	9 feet.	
1 –1.5	0.012	0.011	0.010	0.011
1.5–2	0.030	0.012	0.012	0.018
2 –2.5	0.070	0.017	0.016	0.034
2.5–3	0.140	0.025	0.020	0.061
3 –3.5	0.230	0.037	0.029	0.098
3.5–4	0.220	0.053	0.037	0.103
4 –4.5	0.200	0.074	0.050	0.108
4.5–5	0.110	0.098	0.062	0.090
5 –5.5	0.053	0.124	0.078	0.085
5.5–6	0.024	0.136	0.096	0.085
6 –6.5	0.011	0.137	0.131	0.091
6.5–7	0.005	0.108	0.155	0.089
7 –7.5	0.002	0.090	0.175	0.089

A somewhat novel design for securing even distribution from fixed nozzles was worked out by one of the authors (R. W. P.) for the plant constructed at Akron, Ohio (Mun. Jour., 1915). Imhoff effluent is applied to the filters from a series of control chambers built at varying elevations above the nozzles, so that by cutting out certain chambers the quantity of sewage discharged at a given elevation above the nozzles can be controlled (see Figs. 104 and 105). When the siphon discharges, the depth of sewage in one row of chambers is 1.92 feet, in another 2.82 feet, in another 3.75 feet, in another 4.66 feet, in another 5.60 feet and in three others 6.50 feet.

The best arrangement of dosing cycle should be determined by careful studies in each particular instance. The number of dosing periods will depend on the size of tank and rate of flow and the relation between rest periods and nozzle display periods will be affected also by the nature of the nozzles. Butterfly

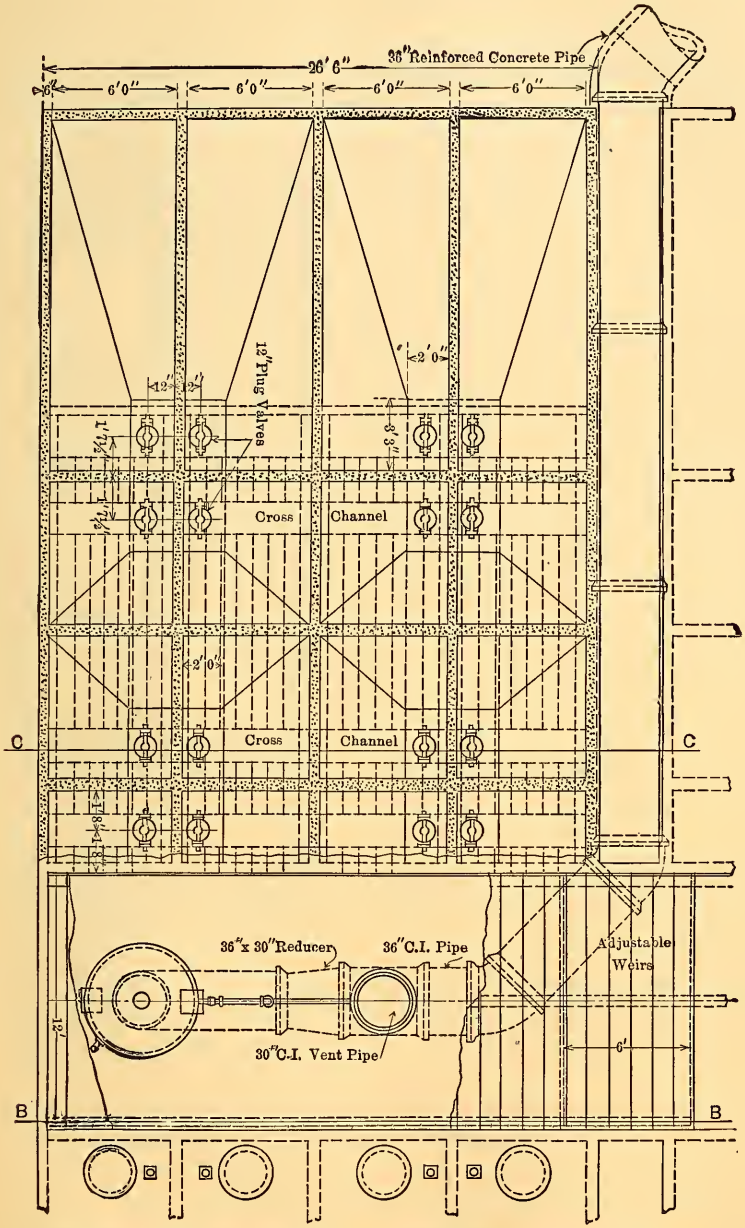

FIG. 104. Plan of Control Chamber for Dosing Trickling Beds at Akron, Ohio.

valves have been used to control the dosing cycle at Baltimore, Reading, and Washington, Pa.

In addition to these minor periods of intermittency, designed merely to promote evenness of distribution, the Columbus in-

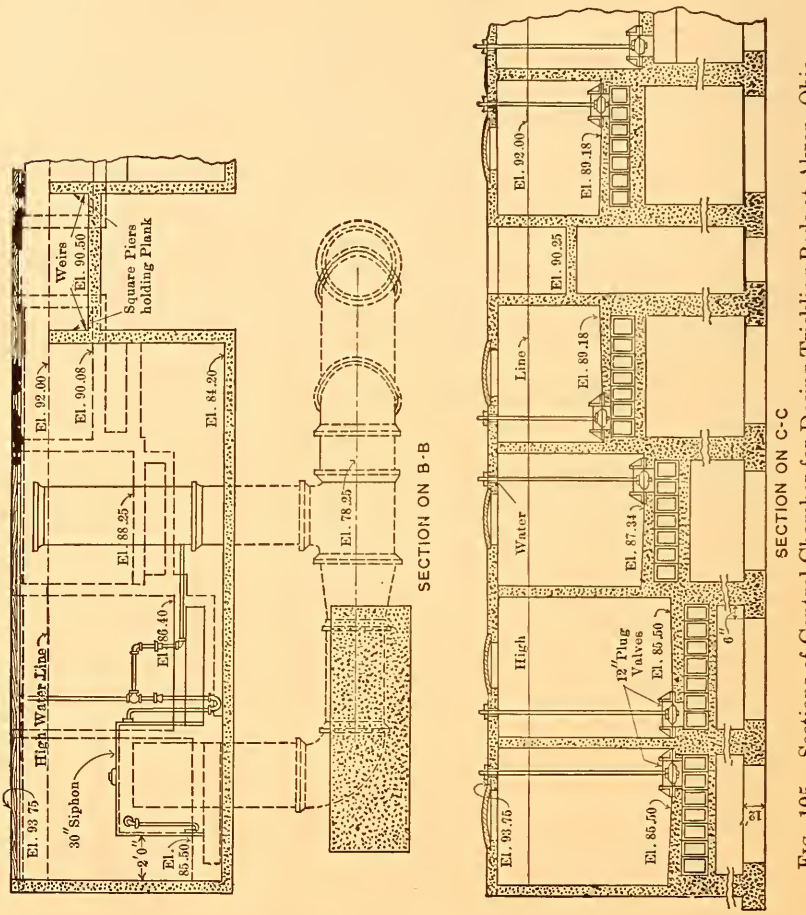

Fig. 105. Sections of Control Chamber for Dosing Trickling Beds at Akron, Ohio.

vestigators recommended longer rest periods for recuperation of the beds. The Massachusetts Institute of Technology experiments on the other hand indicated that steady and continuous operation was beneficial rather than otherwise.

Square-Spray Distributors. W. Gavin Taylor, in his experiments at Waterbury, Conn., devised an ingenious nozzle designed

to cover a square field and thus do away with the waste area between sprinklers. The nozzle, shown in Fig. 106, has an inch opening, and through its center passes a standard which supports a conical spreader, with a vertical cut in each quadrant of the cone so that its upper surface has the shape of a clover leaf. Other types of square spray nozzles have been devised by Merritt at Baltimore and by Chase at Reading, Pa.

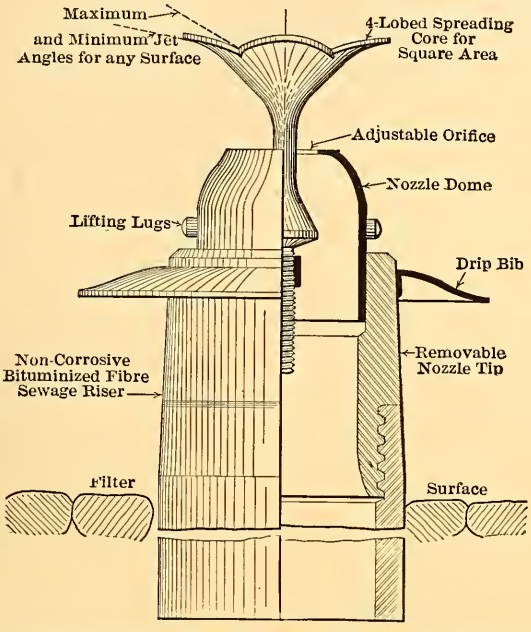

FIG. 106. The Taylor Nozzle for Covering a Square Area (Taylor, 1909c).

The Taylor nozzle was designed to be operated under varying head, so that the sheet spreads in and out from the center. Taylor points out that a straight line variation in head could not be expected to produce ideal results. Theoretically the head upon the nozzle should vary more rapidly near the maximum point than at the minimum; and Taylor (1909c) has designed a pressure undulating valve to secure this end on a practical scale. In Fig. 103 he shows the results obtained with his nozzles (Types L and M) under the ideal pressure variation, under the variation in head obtained by discharge from a straight-sided siphon tank

and under constant head, with similar curves for the Birmingham and Salford nozzles working under a constant head.

Choice of a Distribution System. The choice of a distribution system lies between the English moving distributors, which give an even discharge but are costly and cumbrous, and the fixed sprinkler nozzles and gravity distributors, which are comparatively cheap and simple in operation but give poorer distribution. It is not certain how serious the results of bad distribution may be in practice. At Birmingham Watson (1907) found that a bed dosed with a revolving Candy distributor gave a purification of 83.6 per cent against 80.4 per cent for a bed dosed with fixed sprinklers — a rather insignificant advantage.

Where fixed nozzle distributors are used the main distributing pipes are sometimes carried in walls built up from the floor of the filter (as at Columbus); while at Fitchburg they are laid in the filtering material itself, close to the surface. Risers for the nozzles themselves should be carried 6 inches to 1 foot above the filtering surface, depending on the type of nozzle in use.

The loss of head in a trickling bed of typical American construction (including the loss in the sprinkler system and in the bed itself) is likely to amount to twelve or fifteen feet.

Results of Trickling Filtration. The general experience of the last ten years in England has shown very clearly that the trickling bed is an efficient and economical method of sewage treatment, and offers a satisfactory process for communities where large areas of good sand are not easily available. The rates attainable are much higher than with any other method of treatment, being from two to four times as high as can be used with contact beds. In England trickling rates generally vary from one to two million gallons per acre per day; in the United States the latter figure is generally taken as a safe minimum. In evidence before the Royal Commission, Ducat and Scott-Moncrieff recommend a rate of 1,200,000; Barwise suggests 1,500,000; Watson gives the figures quoted in Table LXXXVII for current English practice. Still higher figures may be sustained for short periods. At Salford the rate, at first 3 million, was raised to 6 million without injuring the quality of the effluent.

TABLE LXXXVII

DEPTH AND RATES OF TRICKLING FILTERS

(Watson, 1903)

Place.	Depth.	Rate (million gallons per acre per day).
Leeds	9	1.2
Accrington	8.5	2.3
Birmingham	5	1.2
Hyde	9	2.6
York	6.5	2.6
Rochdale	9	2.3

The data collected by the Royal Commission may be expressed in tabular form as follows:

TABLE LXXXVIII

RATE OF OPERATION. ENGLISH TRICKLING FILTERS

(R. S. C., 1908)

Rate in million gallons per acre per day.	Number of filters in each class.	Rate in million gallons per acre per day.	Number of filters in each class.
0.5–1	6	2 –2.5	3
1 –1.5	5	2.5–3	3
1.5–2	6	3.5–4	1

The British Royal Commission (R. S. C., 1908) state that a cubic foot of material in the form of a trickling bed will generally treat about twice as much clarified sewage as it will when used in a contact bed. Furthermore the trickling effluent is likely to be better aerated and more uniform in character. The Commission estimates the capacity of trickling beds of coarse material at 100–200 U. S. gallons per cubic yard of filtering material per day, which would amount to a rate of 1–2 million gallons per acre per day on a bed 6 feet deep. The latter figure is equivalent to the treatment by each cubic yard of material of about its own volume of sewage daily. Metcalf and Eddy state that 2000 persons per acre foot is a safe limit for a bed 5–10 feet deep composed of broken stone between 1 and 2 inches in size.

The analytical data for a number of English trickling filters are brought together in Table LXXXIX; and results from various American installations will be cited below. It will be

noticed that the process here is a true nitrification, producing considerable amounts of nitrate in the effluent. The purification is good, distinctly higher in general than that obtained by the double-contact process, but not equal to that produced by intermittent filtration. In cold weather a distinct deterioration in the character of trickling effluents has been noted at Worcester, Mass., and this should be expected from our knowledge of the effect of winter weather on intermittent filters.

TABLE LXXXIX

EFFICIENCY OF TRICKLING FILTERS

Parts per million. (Winslow and Phelps, 1906)

Place.	Material.	Solids.		Nitrogen as —				Oxygen consumed in 4 hours at 80° F.
		Total.	Suspended.	Ammonia N.	Albuminoid N.	Nitrates.	Nitrites and nitrates.	
Accrington *	Sewage	...	....	....	4.6	...	....	49.9
	Effluent	...	....	....	1.5	...	23.3	18.1
Hendon **	Sewage	...	....	71.6	13.2	...	....	147
	Effluent	...	....	2.5	0.8	...	4.8	7.8
Hyde	Sewage	...	....	39.5	16.5	...	....	114
	Effluent	...	....	5.1	1.6	12	13.7	16.3
Leeds †	Sewage	1120	187	21.2	5.1	...	....	57.5
	Effluent	1000	80	8.1	‡1.3	7.8	....	‡9.8
Leeds §	Sewage	1110	229	21.7	5.4	...	....	59.7
	Effluent	1010	110	6.2	‡0.9	9.6	....	‡8.4
Leeds §§	Sewage	1850	768	33.9	12.8	...	....	114
	Effluent	1020	....	11.7	1.4	4.5	....	‡10.1
Leeds ‖	Sewage	1820	850	32.8	12.6	...	....	141
	Effluent	986	....	1.9	0.5	12.1	....	‡3.4
Leeds ‖ ‖	Sewage	1470	486	23.5	9.4	...	....	116
	Effluent	979	81	3.2	1.3	6.2	....	12.1
Wolverhampton ¶	Sewage	...	....	47.1	3.3	1.4	....	43.3
	Effluent	...	....	23.8	0.6	16.4	....	3.6
York ¶ ¶	Sewage	840	....	31.8	5.9	...	....	42
	Effluent	719	....	2.1	0.6	113	....	6.6

* Thermal aerobic filter, September 19 to October 19, 1898, receiving septic effluent (Rideal, 1901).

** Ducat filter, October 14, 1898, receiving crude sewage, single analysis (Rideal, 1901).

† Whittaker bed No. 1, March 9, 1899, to May 8, 1900, receiving septic effluent (Martin, 1905).

‡ Analysis made of the rough settling of suspended solids.

§ Whittaker bed No. 2, September 2, 1899, to January 30, 1900, receiving septic effluent (Martin, 1905).

§§ Ducat filter, March 29 to April 30, 1900, receiving crude sewage (Martin, 1905).

‖ Ducat filter, June 13 to July 7, 1900, receiving crude sewage (Martin, 1905).

‖ ‖ Leeds filter, December 13, 1900, to January 14, 1901, receiving crude sewage (Martin, 1905).

¶ Coal filter, January, 1896, to September, 1898, receiving chemical effluent (R. S. C., 1902).

¶¶ Septic effluent.

TABLE XC

COMPARATIVE STABILITY OF STORED EFFLUENTS FROM CONTACT AND TRICKLING FILTERS

(Clark, 1902)

Time elapsed (days).	Nitrogen as —					Oxygen consumed in 2 minutes boiling, corrected for nitrites.	Oxygen dissolved (per cent of saturation).
	Ammonia N.	Albuminoid N.		Nitrates.	Nitrites.		
		Total.	In solution.				
	Parts per million.						
Trickling Filter No. 135.							
0	20.1	2.3	1	51.8	0.1	24.4	34.3
7	18.2	2.2	0.8	44.2	6	19.5	15.3
14	19.4	1.9	0.7	49.1	1.1	20.9	9.9
21	19.4	2.1	0.8	46.2	1.3	21.5	13.2
28	20.1	2	0.7	39.4	2	21.3	7.7
Trickling Filter No. 136.							
0	13.2	2.4	1.1	52.8	0.1	25.6	51.7
7	13.2	2.1	0.7	50.8	1.1	19.4	15.5
14	13.2	1.9	0.6	49.7	0.5	18.3	5
21	13.2	1.8	0.6	47.3	0.2	17.1	1.6
28	13.5	1.9	0.6	44	0	18.4	0.3
Contact Filter No. 137.							
0	21.4	2	1.4	6.2	0.2	17.4	0
7	23.1	1.8	1.1	0.1	0	22.8	0
14	23.9	1.6	0.8	0.1	0	24.8	0
21	26.4	1.4	0.8	0.1	0	23.2	0
28	26.4	1.3	0.9	0.1	0	23.2	0
Contact Filter No. 163.							
0	14.8	1.6	1	14.9	0.1	11.8	46.2
7	14.8	1.3	0.6	6.6	0	10.8	0
14	15.1	1.1	0.6	5.3	0	10	0
21	16.2	1	0.6	1.6	0	9.4	0
28	17.4	0.9	0.4	2.5	0.2	8.6	0

It must be remembered that the analytical results tell only a small part of the story in the case of the trickling filter. A trickling effluent containing the same amount of ammonia nitrogen and organic nitrogen as a contact effluent will be far less putrescible, since the trickling filter effluent contains a large amount of oxygen, as dissolved oxygen and as nitrates, and this oxygen is available to meet the requirements of the reducing bodies still present. An experiment made by Clark (1902) illustrates this

point. Two trickling effluents and two contact effluents were kept in the laboratory for a month in stoppered bottles, samples being withdrawn every week for analysis. The results are shown in the table on page 355. Initially the organic content of each pair of samples was about the same; but the trickling effluents contained large amounts of nitrates. One contact effluent contained dissolved oxygen at the start; but it lost it in a week and the nitrates fell to a low figure. In the trickling effluents, on the other hand, the dissolved oxygen was not quite exhausted even after a month, and a large fraction of the nitrates remained untouched. The contact effluents were putrefying and the trickling effluents were stable.

The success of the trickling filter must therefore be judged primarily by the stability of its effluent; and the results effected as a filter gradually ripens and becomes efficient may be illustrated by the results tabulated below from experiments at the Technology experiment station at Boston. The distribution upon this particular bed was quite imperfect until the fall of 1906.

TABLE XCI

STABILITY OF TRICKLING FILTER EFFLUENTS

Percentage of samples decolorized at each specified period (see Chap. XVII for significance of this test)

Days.....		0–2	2–4	4–6	6–8	8–10	10–12	12–14	14+	Total samples.
Year.	Quarter.									
1906.....	1	40.5	18.9	2.7	5.4	8.1	0	0	24.4	37
1906.....	2	42.6	26	20.6	2.7	2.7	2.7	0	2.7	73
1906.....	3	23.7	38.9	15.3	5.1	6.8	0	0	10.2	59
1906.....	4 *a*	2.4	9.5	14.3	4.8	9.5	7.1	2.4	50	42
1906.....	4 *b*	2.9	11.4	2.9	8.6	11.4	2.9	8.6	51.3	35
1907.....	1	2.7	4.1	6.7	2.7	2.7	4.1	8.1	68.9	74
1907.....	2	4.4	2.9	1.5	0	1.5	2.9	1.5	85.3	68

Behavior of Suspended Solids in the Trickling Bed. One most important advantage of the trickling process is the comparative freedom of the beds from clogging, due to the fact that their open construction permits whatever solid matter temporarily accumulates to scale off and pass out with the effluent. When a bed is first put in operation there is a period of simple mechanical retention. Thus during the first week of operation the experimental filters at Philadelphia retained between 0.32 and 0.82 pound of suspended solids per cubic yard of filtering medium. Later organic growths began to come through, almost

but not quite balancing the suspended solids applied. After six months of operation 31–160 pounds of material had been stored per cubic yard of filtering medium, giving a reduction of from 0.8 per cent to 7.2 of the open space in the filter. After a year conditions were about the same. Analyses of the deposit showed variations in water content between 48 and 78 per cent; on a dry basis 7.4 to 35.9 per cent was volatile, 0.5 to 9.5 per cent was fat and 0.3 to 1.8 per cent was nitrogen.

A very interesting study was made at Worcester of the fate of suspended solids in a trickling bed the effluent from which was later settled in a humus tank for 1.8 hours. It will be noted that while there was an appreciable destruction of suspended organic matter, mineral solids and iron in suspension were materially increased.

TABLE XCII

FATE OF SUSPENDED SOLIDS IN TRICKLING FILTRATION (½-INCH TO 1½-INCH MATERIAL) FOLLOWED BY SEDIMENTATION

(Worcester, 1913.)

	Pounds per million gallons sewage.				
	Removed from sewage.	Deposited in humus tanks.	Presumably retained by bed.	Stored in bed at end.	Destroyed in bed.
Total suspended solids..	864	756	108	89	19
Organic solids.....	541	351	190	49	141
Mineral solids.....	323	405	−82	40	−122
Suspended iron.....	60	127	−67	16	− 83
Organic nitrogen.....	64	22	42	3	39

Any excess of solid matter which accumulates in the trickling bed is usually discharged in the spring period of unloading discussed on page 322; and it has been pointed out that in this case the annual output of suspended solids was practically equal to the amount applied. The same thing has been found true in actual practice at the Birmingham plant. In case this scaling process does not naturally take place, it should be accelerated by short resting periods, in the course of which the surface films dry out and become detached.

With filters of fine material (¼-to-⅝ inch) clogging takes place at or near the surface, so that surface renewal may be required. The Royal Commission found "examples of this at Salford, Hanley, Chesterfield, Birmingham and Hendon, though at none

of these places was the choking serious. At Market Drayton and Leeds, however, the upper portions of the filters had to be washed or renewed after about three years' work." With coarser

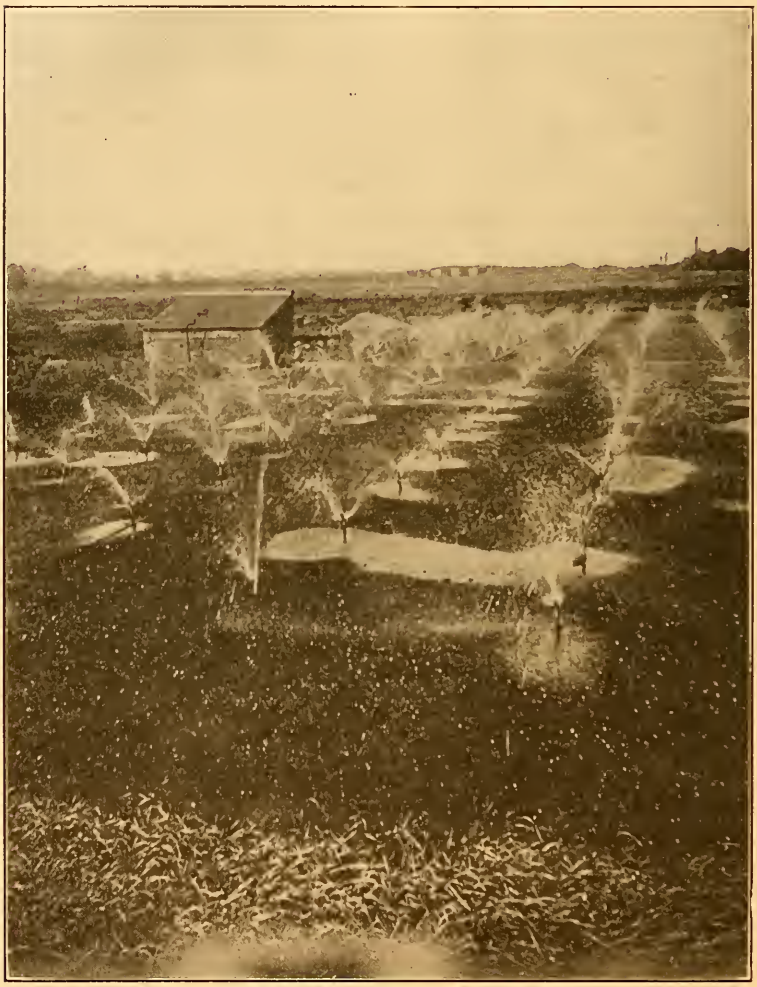

FIG. 107. View of Clogged and Pooling Trickling Filter.

grain filling, on the other hand, trickling beds may be practically permanent. To quote again from the final report of the Royal Commission, " When coarse material is used in a percolating filter, there is apparently little danger of the filter be-

coming clogged, unless the sewage contains much fibre or is liable to give rise to fungoid growths, or unless serious disintegration of the material takes place. We have had no experience of such a filter becoming clogged with suspended matter when dealing with domestic sewage. At Accrington, coarse percolating filters have been in use for the treatment of septic tank liquor for eight years, and have not clogged. At Horfield, a coarse percolating filter treating chemical precipitation effluent and fed by means of dripping trays ran uninterruptedly until it was dismantled after five years' work. At York, coarse filters have been in use for the treatment of septic tank liquor, without interruption, for four years. There are, in fact, a number of such instances. This kind of filter, however, seems to be liable to choke as the result of surface growths. Where there is much fiber in the liquid to be treated (and the same thing would no doubt apply to grease and various trade wastes), percolating filters of even very coarse material may choke up. At Leeds, a percolating bed, which was constructed of coarse coke, showed signs of becoming choked after having received septic tank liquor for eight and a half months. The primary cause of choking was, we think, the fibrous character of the suspended matter, but there was a considerable development of growth on the surface of this bed at the same time."

The causes which lead to fungous growths on the surface of trickling beds are not at present understood; but according to experience at Dorking by Houston and Colin Frye they can be easily eliminated by the application to the surface of the filter of a 20 per cent solution of caustic soda (one gallon to 6.4 square yards,(R. S. C., 1908).

Five methods were used to relieve clogging due to fungous surface growths on the outdoor experimental filters at Philadelphia; resting the beds and then washing out by the ordinary discharge from the nozzle; washing the filter with a fire hose; applying dry bleaching powder to the surface; applying a strong solution of bleach through the nozzles; and continuous disinfection of the influent. The first procedure worked well during the summer but not in winter. The application of dry bleaching powder was not economical, as much of the available chlorine passed through the beds without being utilized. Washing with a fire hose using 115,000 gallons of water per acre proved suc-

cessful and the application (through the nozzles) of a strong solution of bleach (two tons of bleach per acre) was effective at very low labor cost. The continuous disinfection of the influent (as proposed by Dr. Rideal) also proved highly satisfactory. A minor nuisance observed at some of the English works (Accrington and Dorking) and at Reading, Pa., and Worcester, Mass., has been the cultivation in coarse trickling beds of vast multitudes of midges and flies. On account of this fact, and on account of the odors evolved from the sewage spray, trickling filters should, as a rule, be at least an eighth to a quarter of a mile from the nearest house or road.

Experience with Trickling Filters in England and Germany. Birmingham, the fourth city in England, with a population of about 950,000 under its main drainage board, is the one place in the world where the trickling filter has been most thoroughly studied. It holds the same relation to this process which Manchester occupies in regard to contact treatment. The sewage of Birmingham, about 1852, was conveyed by a main sewer into the River Tame, where it was discharged untreated. In 1859 experiments were begun at Saltley which led, in 1872, to the installation of chemical precipitation tanks and a sewage farm. In 1905 the total area of land in the hands of the Drainage Board was 2830 acres, of which 1784 were used for irrigation, and 35 acres for sludge disposal.

On the appointment of John D. Watson, M. Inst. C. E., as engineer to the Birmingham, Tame and Rea District Drainage Board, he began an elaborate series of large-scale experiments which, unfortunately, have never been reported fully in print. The results of these experiments have led to the entire abandonment of chemical precipitation and sewage farming at Birmingham, and to the treatment of the sewage on a strictly biological basis. In 1901 a beginning was made by converting the precipitation basins into septic tanks, and in 1903 the first trickling beds were built. Finally the flow of about 30 million gallons was treated in a plant including the following elements: (*a*) sedimentation or detritus tanks; (*b*) septic tanks; (*c*) secondary sedimentation or silt tanks; (*d*) trickling beds; (*e*) separating tanks for removing suspended solids from the tricking effluent.

The Birmingham plant was described by Watson (1910) as follows: The detritus tanks are 5 in number and have a total

capacity of 500,000 gallons. The first septic tanks have a capacity of 6,730,000 U. S. gallons, a mean rate of flow of 1.2 feet per minute and a sedimentation period of 4.36 hours. The second septic tanks are 20 in number with a total capacity of 8,700,000 U. S. gallons, the storage period being 8 hours. These preliminary works are at Saltley, and here also are 30 acres of bacteria beds to be used chiefly as storm water beds. These are 6-foot trickling filters filled with clinker. From Saltley the septic effluent flows for 5 miles through a closed conduit to Minworth in the township of Sutton Coldfield where the trickling beds have been built. Here the septic effluent is first treated in the silt tanks mentioned above to reduce still further its suspended solids. These tanks (see Chap. IV) are 22 in number; the first six built are circular, 44 feet in diameter and 33.5 feet from the coping level to the bottom of the sump, the lower portion being in the form of an inverted cone having a batter of 1 to 1; the sixteen newer ones are 25 feet square and 20 feet deep to the apex of the pyramid. The septic effluent enters these newer tanks by a pipe dipping down to the middle of the tank, with a downward velocity of 1–2 feet per second. As it emerges from the mouth of the pipe it spreads out and ascends to the square portion of the tank at a decreasing velocity, which averages 7 feet per hour. The storage period in these tanks is about 4 hours. The sludge removed from the bottom amounts to 1918 pounds of dry matter per million United States gallons of sewage treated and the cost of removing and burying it is about $14\frac{1}{3}$ cents per million gallons of sewage treated (Watson, 1907). The tanks are capable of effecting a reduction in suspended solids from 291 parts per million (the septic effluent value) to 61 parts per million; and as a matter of fact they did reduce the suspended matter in 1907 from 242 to 97 parts; and the total cost is about 20 cents per million gallons.

The total amount of dry solid matter removed from the sewage is 67 long tons per day, or 1170 tons of liquid sludge containing 94.5 per cent water.

At first the sludge was spread in great shallow lagoons and allowed to dry sufficiently to be dug into the ground. This method was changed by discharging the sludge into trenches 3 feet wide and 18 inches deep, and covering it over with earth as soon as practicable. In 1907 the sludge was pumped to a

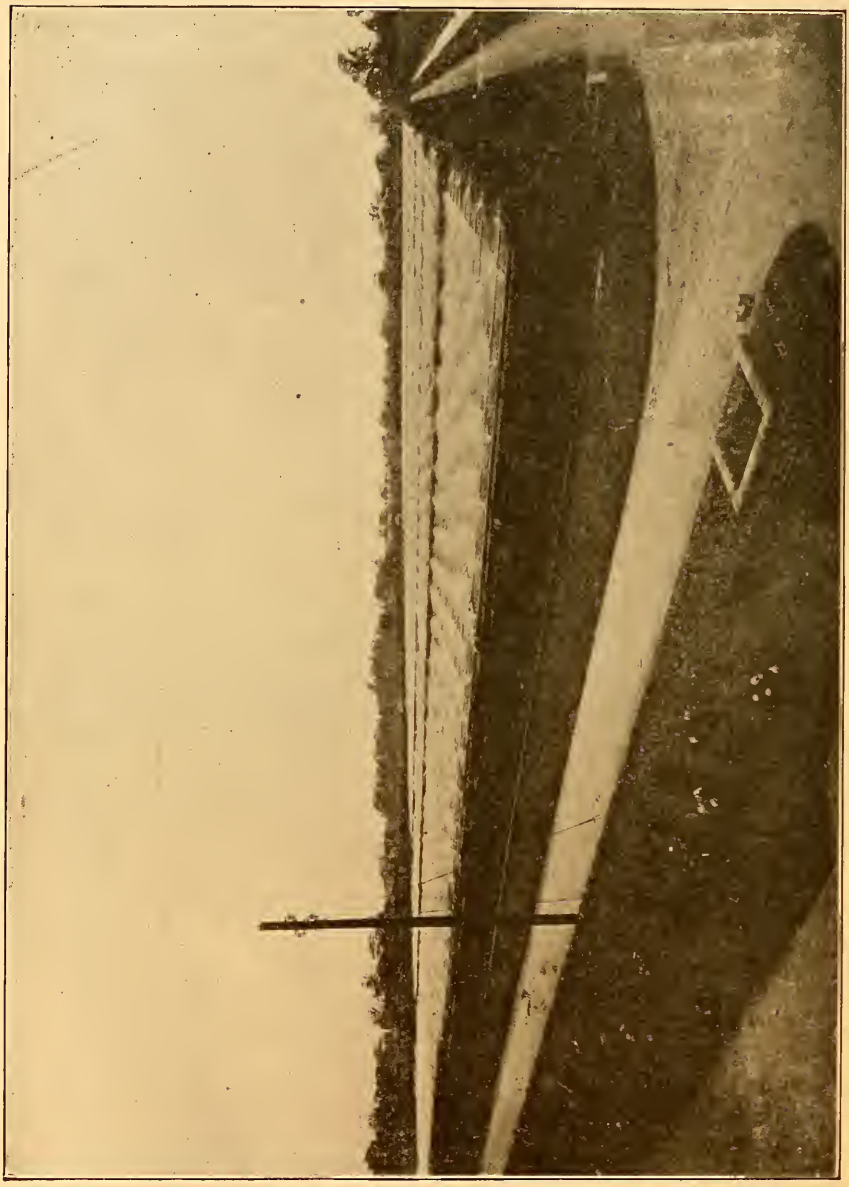

FIG. 108. General View of Birmingham Filters (courtesy of J. D. Watson).

field surrounded by earthen embankments 10–20 feet high, and although about 2 acres of this area were covered with sludge to a depth of 8 feet, and rapid decomposition of the organic matter was going on, as shown by the amount of gas continually evolved, no strong odor was noticeable from the adjoining banks.

The trickling beds at Sutton Coldfield occupy 30 acres. The beds are 1, 2 and 3 acres in area, with the exception of some quarter-acre and half-acre beds built for experimental purposes. The larger filters are rectangular in shape, with side walls of rubble, laid dry, and floors of concrete, with a fall of about 9 inches across the bed. Over the concrete is an aerating floor of semicircular stone-ware tile, laid with loose joints. The filling material is 6–7 feet deep and consists of broken brick, slag, granite or quartzite $\frac{3}{4}$ to 2 inches in diameter. Watson's experiments led him to the conclusion that the nature of the filling used was immaterial, so long as it was permanent and had a rough surface. Fine material, of course, gave somewhat better results; but the effluent from beds filled with 2-inch material was nonputrescible and the cost of operation materially less. Thus the operation of a bed mainly of $\frac{1}{4}$–$\frac{1}{2}$-inch material with 1 foot of $\frac{1}{8}$–$\frac{1}{4}$-inch material on top and 2.25 feet of $1\frac{1}{2}$–$2\frac{1}{2}$-inch material below cost $13.60 per million United States gallons against $7.95 for a bed of $1\frac{1}{4}$–$2\frac{1}{2}$-inch material. The average rate of operation is 900,000 United States gallons per acre per day. Distribution is effected by means of the fixed sprinkler described on page 339. A general view of one of the beds is shown in Fig. 108, and another view in winter in Fig. 109.

After filtration the trickling effluent contains about 125 parts per million of solids. This is treated in separator tanks at a cost of 28 cents per million United States gallons to eliminate solids; pumping sludge from these tanks and digging it into the land costs ten cents more (Watson, 1907).

Analytical results for the various parts of the Birmingham plant, in 1909, are given in the table on page 364.

FIG. 109. View of Birmingham Beds in Winter (courtesy of J. D. Watson).

TABLE XCIII

COMPOSITION OF SEWAGE AND EFFLUENTS AT BIRMINGHAM

Analyses of average weekly samples collected every half-hour during the 24 hours, in the year 1909.

Parts per million.

	Crude sewage.	Septic effluent.	Silt tank effluent.	Trickling effluent.	Separator tank effluent.
Oxygen consumed in 4 hours unfiltered	277.4	200.4	113.2	39.9	21.3
Albuminoid nitrogen	12.5	9	5.9	5.5	1.8
Nitrogen as nitrates and nitrites	....	....	....	11.3	15.7
Suspended solids	408	194	98	128	15

The average capital cost of the last eighteen beds built at Birmingham was about $35,000 and the total cost of treatment taking the larger beds with their appurtenant tanks, etc., as a basis, and including all interest charges and operating expenses averages about $6.80, per million U. S. gallons.

Important investigations of the trickling process have been made at many other English cities besides Birmingham, the most notable of which, perhaps, are the studies at York (York,

1901). At several other large cities and at many smaller places, trickling filters are in operation. Baker (1904) gives good descriptions of plants of this type at Salford, Accrington, and York, and Clark (1908) describes a visit to the filters at Blackburn, Haywood, Chesterfield, and Hanley. At Blackburn the use of soft material had led to disintegration and clogging, but at the other three plants excellent results were reported.

In Germany, too, the trickling filter has been recognized as a most promising biological method. One of the most interesting

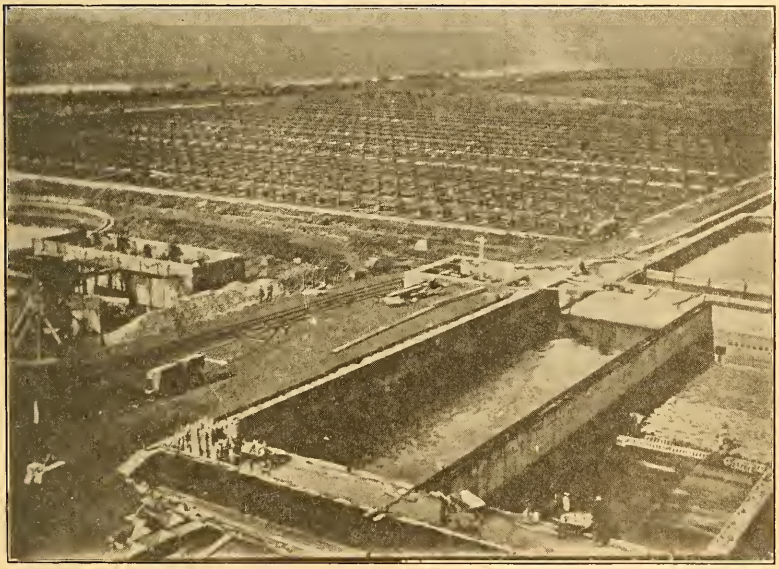

Fig. 110. General View of Salford Beds during Construction.

trickling filter installations in this country was constructed at Wilmersdorf, a suburb of Berlin. The plant is designed for a maximum flow of 38,000,000 gallons per day, and includes septic tanks, trickling beds, secondary sedimentation basins for the trickling effluent and accessory sand filters. The septic tanks are six in number and have a total capacity of 3,178,000 gallons. The septic effluent passes to a gathering chamber, from which it is discharged automatically on the beds. The latter are 56 in number, circular filters, each 65.6 feet in diameter. They are built on concrete floors but without walls, and are dosed by

revolving sprinklers of the Barker Mill type. Figure 111, showing these beds, is from a detailed description of the plant by Gerhard (1908).

Early Studies of Trickling Filters in the United States. The general recognition of the trickling filter as a practical method of purifying sewage in the United States really dates from the experiments carried out by the city of Columbus, Ohio, in the years 1904 and 1905. A few small plants were in operation before that date (at Madison and West Allis, Wis., for example), but it was still uncertain to what extent the trickling process would operate successfully in the severe winter climate of the northern United States.

Columbus at this time was a city of 125,000 inhabitants discharging its sewage into the Scioto River, a small stream quite incapable of handling it satisfactorily. Double filtration through coke had been suggested in 1898, septic treatment in 1900, and intermittent sand filters in 1901. In 1903, Hering and Fuller of New York were called in consultation, and an experimental study of the problem was begun in the spring of 1904. The experiment station was under the immediate charge of G. A. Johnson. It included 7 tanks for preliminary treatment, 2 coke strainers, 6 contact beds, 6 trickling beds and 21 intermittent sand filters. These were studied with care during a period of one year's operation, and the results were discussed in an exhaustive report (Johnson, 1905). The final conclusion was that the trickling process could be operated with perfect success under the local conditions and that it would furnish the best solution of the problem.

At about the same time that this work was completed an outdoor trickling filter, 200 square feet in area, was put in operation at the Boston experiment station of the Massachusetts Institute of Technology, and two years later it was concluded that this process would offer the best means of treating sewage similar in character to Boston sewage (Winslow and Phelps, 1907). In 1905, an experiment station was designed for the city of Waterbury, Conn., under the direction of R. A. Cairns, and a valuable series of experiments was carried out by W. Gavin Taylor (Taylor, 1907*b*). A similar experimental plant was built at Baltimore, Md., in 1907 (Eng. News, 1907), under the direction of Calvin W. Hendricks and in immediate charge of E. B.

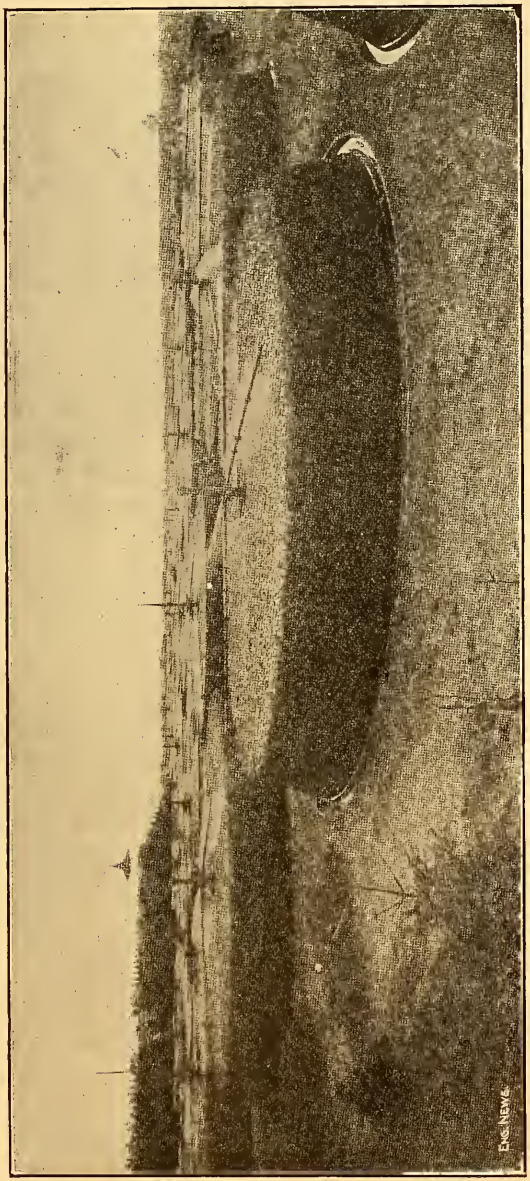

FIG. 111. Trickling Filters at Wilmersdorf, Germany (Gerhard, 1908).

Whitman. In both of these investigations the trickling filter proved its value.

Exhaustive and important studies of this process of sewage treatment were conducted at Worcester, Mass., in 1911–1912. The beds were of considerable size (each 550 square feet in area) and they were operated for 18 months. The filtering material in one bed was of $\frac{3}{4}$-inch to $2\frac{1}{2}$-inch crushed stone and in the second bed of $\frac{1}{2}$-inch to $1\frac{1}{2}$-inch crushed stone. The depth of each bed was 10 feet and they were dosed by the Worcester nozzle (described above), operated under a head varying between 8 feet and 2 feet. The rate of application was gradually increased from one to two million gallons per acre per day, averaging 1,517,000 gallons for the period of 18 months. The nozzles were remarkably free from clogging but considerable trouble was experienced due to pooling caused by organic growths on the surface of the beds. Great numbers of moth flies (*Psychoda alternata*) bred in the beds and fish-worms were very abundant. Effluents were consistently stable except during cold weather; those which fell below 5 parts per million of nitrate nitrogen were generally found to be unsatisfactory. The general results of the treatment (including secondary sedimentation in humus tanks) are indicated in the table below.

TABLE XCIV

EFFICIENCY OF TREATMENT IN TEN-FOOT TRICKLING FILTERS FOLLOWED BY SEDIMENTATION FOR 2.2 HOURS AT WORCESTER, MASS.

(Worcester, 1913.)

	Parts per million.								
	Nitrogen.				Total suspended solids.	Oxygen consumed.	Dissolved oxygen (per cent saturation).	Bacteria per c.c. 20°.	Per cent of samples non-putrescible.
	Total organic.	Ammonia N.	Albuminoid N.	Nitrates.					
Applied sewage.	41.7	24.3	6.7	0.7	144	119	.....	.....	5.5
Effluent of coarse bed.....	24	18.5	2.5	5.1	45	31	52	1,260,000	84.8
Effluent of fine bed.....	22.9	17.8	2.3	6	43	29	64	890,000	88.4

Trickling Filter Installations at Reading, Washington, Columbus, Akron, Atlanta and other American Cities. The first large trickling plant actually constructed in this country was built at Reading, Pa., by Hering and Fuller in co-operation with the city

Engineer, Elmer H. Beard, and O. M. Weand, and was put in operation in February, 1908. The plant has a capacity of 2 million gallons a day, and includes careful preliminary screening, septic treatment, trickling filters and final sedimentation. Screening is made particularly thorough on account of the fibrous material contributed by certain factory wastes. The device in use has already been described in Chapter III. After screening, the sewage is pumped to the septic tank on the opposite side of the Schuylkill River, a distance of 6600 feet. The septic tank is open and of concrete 51 feet 8 inches wide, 253 feet long and 16 feet deep to the flow-line, the unusual depth being due to the fact that the steel framework of an earlier disposal plant was used in its construction. The capacity of the tank is 1,600,000 gallons, or 19 hours' flow. The influent enters the tank from twelve 5-inch holes in a 14-inch steel pipe running across one end; the effluent passes over a weir at the other end. Five inches in front of the weir a scum-board extends 1 foot below the crest of the weir.

The trickling filters, two one-acre beds, are on Fritz's Island in the Schuylkill River, 2500 feet from the septic tank. The filling material is blast furnace slag, broken by hand to a size between $1\frac{1}{2}$ and 4 inches. The outside of the filters is formed of vertical walls of dry rubble for a short distance up, and above, of large pieces of slag laid by hand to a batter of about 2 horizontal to 3 vertical. The depth of the filters is 5 feet. The sprinkling nozzles are of the general Columbus type, but modified by supporting the spreading cone on a rod passing up through the orifice (as in the Birmingham sprinkler), thus avoiding the interference due to the supporting side arms of the Columbus nozzle. The cone has 45 degree sides; the supporting rod has a diameter of $\frac{5}{16}$ inch and the orifice itself is $\frac{5}{16}$ inch in diameter. The sprinkler heads are set at intervals of 14 feet 2 inches on parallel lines of 8-inch pipes, laid 13.7 feet apart a short distance above the floor of the filter. The distribution system is dosed from a siphon tank, so arranged that the nozzles work under a head varying from 6 feet to 1.4 feet, a complete cycle occupying about 11 minutes.

The effluent from the trickling beds is settled in a basin 100 feet by 95 feet 9 inches in area and varying in depth from 4 to 5 feet. The capacity is 340,000 gallons, or one-ninth of the daily flow. The analytical results for the 14 months, February, 1908, to March, 1909, are given in Table XCV.

TABLE XCV

ANALYSES OF SEWAGE AND EFFLUENT AT READING

Parts per million. (Fuller, 1909.)

	Suspended solids.	Oxygen consumed.	Nitrogen as nitrates.	Dissolved oxygen.	Bacteria per c.c.
Screened sewage	165	57	.....	.....	3,100,000
Septic effluent	43	26	.....	.....	1,800,000
Filter effluent	.....	.....	4.5	.....	600,000
Settled effluent	20	15	5	6.5	670,000

The second large trickling filter in the United States was completed in the fall of 1908 at Washington, Pa. The dry-weather flow is about one million gallons per day, and the sewage from 1904 to 1908 was very imperfectly treated in a septic tank and crude stone strainer. The new plant, capacity two million gallons, was designed by one of the authors (R. W. P.) and was built under his supervision by D. M. Belcher as Resident Engineer. It includes as its main elements a screen chamber with two inclined screens having $\frac{5}{8}$- and $\frac{1}{4}$ inch mesh, described in Chapter III; four open concrete septic tanks with a total capacity of 800,000 gallons; trickling filters; and a final sedimentation basin. The four filters are each 100 by 150 feet in superficial area (see general plan in Fig. 112). Walls and floor are of concrete, and the filtering layer consists of 6 feet 10 inches of broken stone between 1 and 4 inches in diameter. The bulk of the bed was made of a low-grade local limestone with 6 inches of harder stone on the top. Sewage is sprayed over the surface by nozzles of the Columbus type, placed 12.5 feet apart on lines 10.75 feet apart. They are operated with a head varying between 7 feet and 2 feet, the average discharge being 10 gallons per minute. The filters are underdrained with half-tile, 61,000 feet of pipe being used for the 1.5 acres of filter.

The plant at Columbus, Ohio, which first went into operation in November, 1908, is one of the largest and most interesting of American trickling filters. The capacity of this plant is 20,000,000 gallons, and it was designed under the direction of Hering and Fuller of New York, consulting engineers, and Julian Griggs of Columbus, chief engineer. John H. Gregory was engineer of design and construction, and he has published an excellent description of the plant (Gregory, 1910).

The sewage is roughly screened at the pumping station by bar screens, and thence passes to the purification works, where it was originally treated in primary and secondary septic tanks (since remodeled as Imhoff tanks). In the controller well is a device for maintaining a constant flow to the filters. The controllers

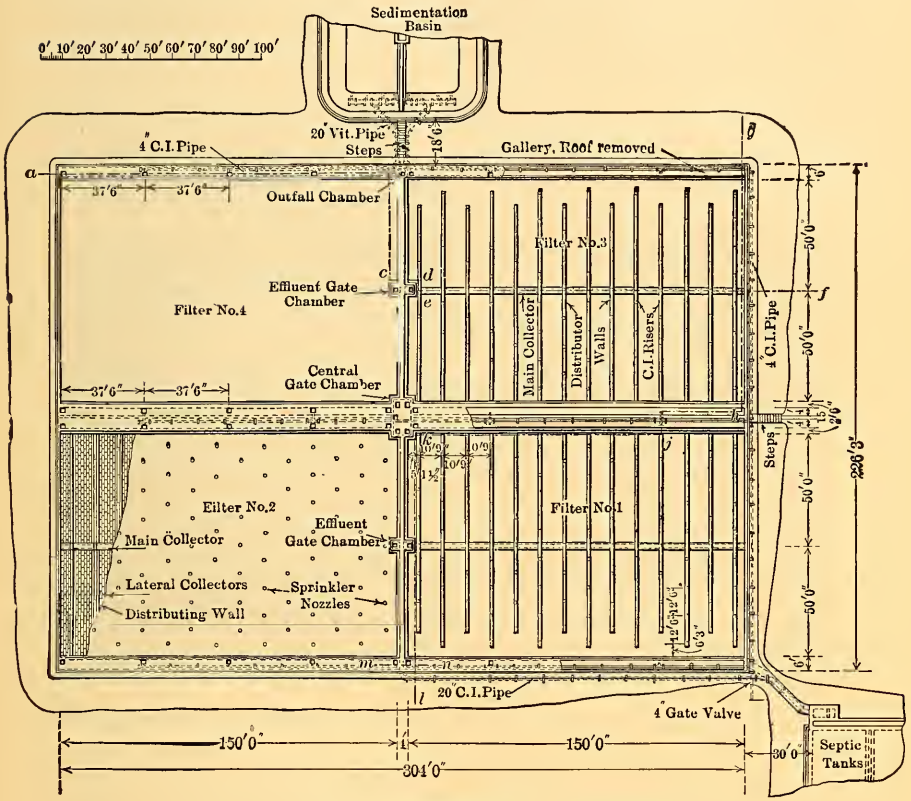

FIG. 112. Plan of Trickling Beds at Washington, Pa.

may be adjusted by a system of weights to deliver any quantity between 10 and 22 million gallons per day. From the controller well the sewage passes by 42-inch gates to the distributing well, and thence by 24 × 36-inch gates into the manhole chambers and filter distributors. The effluent passes from the main collectors into the sump wells, and thence into the effluent well and the effluent conduit. There are in all forty-two sluice gates, all operated by hand.

The six filters, four of which have been constructed for present use, are designed as equilateral triangles about 500 feet on a side, radiating out from the central gatehouse. Each filter has an area of 2.5 acres, and all are constructed of concrete and steel. Sewage is distributed to each half-filter by a 30-inch main of reinforced concrete supported by cross walls which carry 5- and 6-inch lateral distributors. These laterals are 13 feet $9\frac{7}{8}$ inches apart and carry risers spaced 15 feet 4 inches between centers.

FIG. 113. View of Columbus Filters during Construction (courtesy of J. H. Gregory).

Each set of three sprinkler nozzles forms an equilateral triangle 15 feet 4 inches on a side. The general arrangement of main and lateral distributors and underdrains is shown in Fig. 113. The sprinkler nozzles are of the general Columbus type, shown in Fig. 97, with $\frac{9}{16}$-nozzle openings. Each orifice is rated to discharge 13.5 gallons per minute under a 5-foot head, and there are 211 nozzles to the acre. In order to secure more even distribution each bed, or half-bed, was dosed in 1909 for successive periods under three different heads 4 feet, 7 feet and 9 feet; and between each period the bed was given a period of entire rest.

The net result of this mode of operation is that the beds are dosed at a rate of about 4 million gallons per acre for half the time, resting for the other moiety.

The filtering layer itself is 5.5 feet deep, the lower 10 inches being 3–4 inch limestone and the rest 1.25–3 inch material of the same sort. 80,125 cubic yards of filling were used, costing in place $1.57 per yard. The floor of the filter is of 4-inch concrete and is practically covered with 6-inch notched half-tile, bedded about $\frac{1}{4}$ inch into the concrete, while it was soft (see Fig. 113).

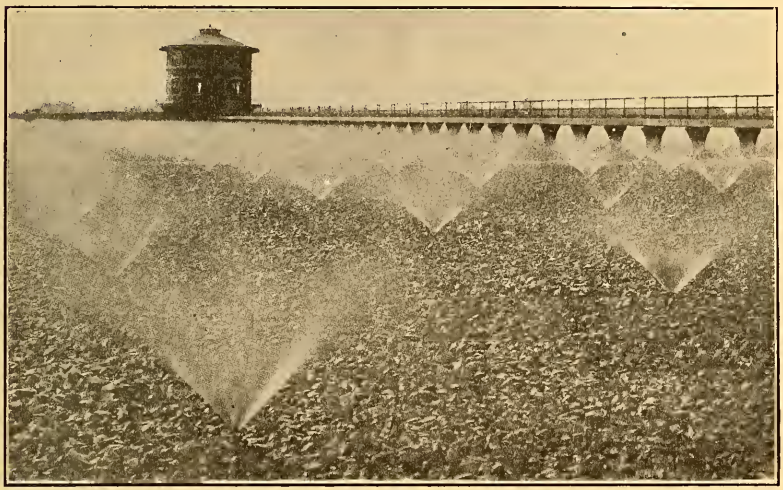

FIG. 114. View of Columbus Filters in Operation (courtesy of J. H. Gregory).

There are about 100 miles of this tile in the entire plant. A general view of one of the filters is shown in Fig. 114.

The effluent from the trickling beds is finally settled in two settling basins, each having a capacity of 2,000,000 gallons. The sludge from these basins is pumped out and discharged into the Scioto River at times of high water; and when the river is above a certain height the trickling effluent itself is discharged directly into the river without preliminary sedimentation.

The principal results of sewage treatment at Columbus for seven years are shown in Table XCVI, taken from the Annual Report of the Division of Sewage Disposal for 1915.

TABLE XCVI

ANALYTICAL SUMMARY COLUMBUS SEWAGE TREATMENT WORKS

Parts per million.

Year.	Suspended matter.				Dissolved oxygen consumed in 24 hours at 37° C.			
	Raw sewage.	Tank eff.	Sprinkler eff.	Final eff.	Raw sewage.	Tank eff.	Sprinkler-eff.	Final eff.
1909	201	82	84	41	....	....	.....	....
1910	211	80	79	50	102	73	23	27
1911	287	99	96	....	131	95	25	....
1912	245	109	88	....	198	146	28	....
1913	300	121	63	65	147	104	26	22
1914	260	104	91	88	131	98	22	21
1915	241	130	93	88	147	119	48	46

Year.	Nitrogen as nitrites and nitrates.	Dissolved oxygen.	Per cent saturation.	Stability value.
	Final eff.	Final eff.	Final eff.	Final eff.
1909	3.35	5.8	.....	89
1910	4	7.7	.....	84
1911	4.60	.....	.....	71
1912	6.10	.....	.....	84
1913	4.10	4.5	47	67
1914	4.70	5.9	56	79
1915	1.80	4.5	49	26

Tank treatment removed only 46 per cent of the suspended solids in 1915 as against an average of about 60 per cent in preceding years. The per cent purification of oxygen consumed by the entire treatment was only 69 per cent in 1915 and the relative stability of the final effluent averaged only 26. The reason for the less satisfactory results in this year was that the septic tanks were being rebuilt on the two-story plan and while one-half of them were in process of reconstruction the total flow of sewage was passed through the other half.

The disposal plant at Akron, Ohio (Mun. Jour., 1915), was constructed in 1915 according to the design of one of the authors (R. W. P.). It included bar screens (1-inch open space), grit chambers, four rectangular Imhoff tanks (each of three sections) control chambers of special design (described and figured on pages 349 and 350), and four trickling beds, each 250 feet long by 175 feet wide. Vertical vent ducts capped by cowls are placed at the upper ends of the effluent channels. Seven sand beds,

each 160 feet long and 21 feet wide, are provided for drying the sludge; and the dried sludge can either be delivered to farmers, removed by rail or mixed with the tankage produced from the garbage reduction plant, located at the same site for this special purpose. The plant was built to serve a population of 100,000.

Other large trickling filter plants have been operated at Baltimore, Md. (50 million gallons), since 1912; at Atlanta, Ga. (three plants with a total capacity of 16 million gallons) since 1912; and at Rochester, N. Y. (33 million gallons), since 1917. The results of treatment at Atlanta have been described by Hommon (1916). The Proctor Creek plant has 1.5 acres of beds designed for 20,000 people, but actually in 1916 treating the sewage of 30,000; the Peachtree Creek plant has 2.5 acres treating the sewage of about 68,000 people; the Intrenchment Creek plant has 2 acres and a contributing population of 50,000. The beds have on the whole yielded excellent results, except for clogging of the surface of the Peachtree Creek plant by algæ in the spring of 1915.

The results at the three plants are given by Hommon for 1916 as follows:

TABLE XCVII

RESULTS OF TANK TREATMENT AND TRICKLING FILTRATION AT ATLANTA

	Ammonia nitrogen.		Organic nitrogen.		Nitrites and nitrates.		Dissolved oxygen, per cent saturation.		Relative stability.
Effluent from.....	Tank.	Filter.	Tank.	Filter.	Tank.	Filter.	Tank.	Filter.	Filter.
Peachtree Creek.....	10.1	3.5	6.1	3	0.6	6.7	6	68	98
Proctor Creek.....	20.6	7.9	4.6	2.8	2.8	9.7	...	...	98
Intrenchment Creek..	11.7	4.3	7.1	3.6	0.4	5.6	...	...	97

The Removal of Suspended Solids from Trickling Filter Effluents. Where a sewage effluent is discharged into a rather large volume of water, and particularly in a region of strong currents, the effluent may be satisfactory just as it comes from the trickling bed. Such is the case at Columbus when the Scioto River is high; and such will often be the case with maritime towns. Where the effluent must be discharged into a small stream or a shallow bay, supplementary sedimentation for the removal of suspended solids is essential. The effluent as a whole may be

stable; yet if the solids settle and accumulate on the bottom of a stream, — by themselves and in the absence of the ample supply of oxygen carried by the liquid, — they may prove putrescible. Secondary sedimentation forms, therefore, an integral part of most trickling filter plants.

Fortunately, the solids in the trickling effluent are in comparatively dense masses and settle out very readily. As a rule, with a two-hour storage period a removal of one-half to two-thirds of the suspended matter present is easily effected. Data for the Birmingham, Columbus, and Reading plants and for the experimental filters at Boston are brought together in the table below.

TABLE XCVIII

SEDIMENTATION OF TRICKLING EFFLUENTS

	Flow period, hours.	Suspended solids, parts per million.	
		Trickling effluent.	Settled effluent.
Birmingham	.....	65	18
Columbus	5.7	84	41
Reading	2.6	.....	20
Boston	2	117	57

The weight of opinion, both in England and in this country, has, in the past, been in favor of securing a fairly complete removal of suspended solids before treatment on trickling beds. Thus there is generally involved a double system of sedimentation, preceding and following treatment. In certain cases, when a rather weak sewage is treated on beds of coarse material, it may be possible to dispense with the preliminary treatment entirely, applying crude sewage to the filters and removing suspended solids only after organic stability has been attained. The table on page 377, for example, shows the result of two years' experiments at Boston. The final effluent from the process, including septic treatment, was of course better than the other, having 50 parts of suspended solids against 65. On the other hand, the ammonia nitrogen was higher in this effluent and the organic nitrogen and oxygen consumed only slightly lower. Both filter effluents, even without sedimentation, were stable for four days 90 per cent of the time.

TABLE XCIX

PURIFICATION OF BOSTON SEWAGE WITH AND WITHOUT SEPTIC TREATMENT, 1905-1907

Parts per million. (Winslow and Phelps, 1907.)

	Turbidity.	Sediment.	Suspended solids.		Nitrogen as —			Oxygen consumed.	Oxygen dissolved.
			Total.	Fixed.	Organic nitrogen.	Ammonia N.	Nitrates.		
Crude sewage.....	279	121	135	44	9.1	13.9	0.2	56	3.4
Trickling effluent...	200	136	138	58	7.1	10.4	4.4	41	7.9
Settled effluent.....	128	62	65	24	4.1	10.2	4.6	31	....
Septic sewage.....	213	87	81	18	6.5	17.5	0	57	0
Trickling effluent...	147	78	96	37	5.8	12.6	4.7	34	7.6
Settled effluent.....	106	51	50	18	3.6	12.6	4.8	29	....

Comparative Costs of Trickling Filtration and Other Processes. In general it seems clear that treatment on trickling beds is likely to prove the most generally economical method of purifying sewage to a point of reasonable organic stability. In construction the great point in favor of the trickling bed is the smaller area required. The table below, from results obtained at Croydon, gives an excellent idea of the comparative efficiency of unit areas:

TABLE C

COMPARATIVE EFFICIENCY OF TRICKLING AND CONTACT BEDS

(Farmer, 1909.)

	Contact beds. 4 years.	Trickling beds, 2 years.
Total gallons treated.....	892,228,000	962,496,000
Gallons per square yard per day.....	61	173
Gallons per cubic yard per day.....	46	104
Per cent reduction oxygen consumed.....	47.3	76.8
Per cent reduction albuminoid nitrogen	40.6	73.8

One acre of intermittent sand filter will handle the sewage from 500–1000 persons; one acre of a double contact system will suffice for 4000–5000 people; while an acre of trickling surface can purify the sewage from a population of at least 10,000. The cost of distributing apparatus is of course a serious con-

sideration in the case of the trickling filter, but this is seldom sufficient to offset the advantage of the greatly diminished area.

In regard to operating cost, the advantage is again generally in favor of the trickling process. Intermittent filters are expensive, if properly cared for, and although the ordinary maintenance of the contact bed may be slightly less than that of the trickling bed, this is outweighed by the liability to washing and replacing filtering material which appears not to be required in the trickling process.

It is difficult to make general statements in regard to costs which are so markedly affected by local conditions as those involved in sewage purification work. Rather careful studies based upon certain detailed assumptions have, however, been made by high authorities, in England and America, which may perhaps be taken as representing the relative expense of contact and trickling treatment.

In the final report of the British Commission on Sewage Disposal (R. S. C., 1908), G. B. Kershaw presents comparative cost data for a plant to treat a dry-weather flow of one million gallons a day of average domestic sewage, which led the Royal Commission to the final conclusion that " on the basis which we have adopted, purification of sewage (after preliminary treatment) by means of percolating filters costs only about two-thirds as much as purification by double contact beds. Where, however, the sewage has first been subjected to quiescent settlement, with chemicals, and single contact is sufficient to produce a satisfactory effluent, the cost becomes more nearly equal, though percolating filters are, even in that case, slightly cheaper."

A somewhat similar idea of comparative costs under American conditions may be obtained from the estimates prepared for the International Waterways Commission by Hering and Fuller of New York in regard to the disposal of the sewage from the Calumet district of Chicago. These figures were made on the basis of a daily flow of 156 million gallons and include estimates for intermittent filtration as well as contact and trickling treatment. The cost of intercepting sewers and pumping stations is included because this factor is directly affected by the method of treatment chosen, intermittent filtration requiring a much longer sewer in order to reach suitable sand areas. In preparing the table below the figures for annual operation given by Hering and Fuller have

been reduced to a million gallon basis and interest on their construction charges has been calculated at 5 per cent. As a matter of fact, the costs should of course be proportionately higher for a small plant than for the one contemplated in the original estimates. The area allowed for 156 million gallons was 1200 acres in the case of the sand filters, 300 acres for contact beds and 80 acres for trickling beds.

TABLE CI

COMPARATIVE COST OF SEWAGE TREATMENT AT CHICAGO BY VARIOUS METHODS

Cost per million gallons. (Hering and Fuller, 1907.)

	Intermittent filters.	Contact beds.	Trickling beds.
Construction charges:			
Intercepting sewers, pumping stations and appurtenances	$4.46	$2.90	$2.90
Septic tanks, covered, including sludge disposal facilities	0.83	0.83	0.83
Filters, office, laboratory, etc	3.16	5.27	3.16
Settling basins	.....	.....	0.18
Contingencies and supervision, 15 per cent	1.27	1.35	1.06
Total	9.72	10.35	8.13
Operation costs:			
Pumping, fuel, labor and repairs	5.26	3.52	3.52
Supervision, analytical and clerical assistance	0.44	0.53	0.53
Care of septic and settling tanks, including sludge disposal	0.63	0.63	0.95
Care of filters	8.42	4.56	1.93
Supplies and miscellaneous	0.44	0.44	0.44
Total	15.19	9.68	7.37
Total cost	24.91	20.03	15.50

Of course the English and American estimates are not directly comparable, since each includes items left out in the other and since the unit costs are widely different. The relative costs of the different processes are, however, strikingly concordant. In each case the cost with contact treatment is between 40 and 50 per cent higher than with trickling beds; and intermittent filtration appears from the Calumet estimates to be still more costly than contact treatment.

Construction costs (actual or estimated) of various American trickling filters per cubic yard of filter were as follows according to Metcalf and Eddy (1916): Columbus, O., $2.81; Washington, Pa., $2.89; Patterson, N. J., $3.14; Baltimore, Md., $3.49;

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Reading, Pa., $3.50; Fitchburg, Mass., $3.70; East Orange, N. J., $3.72; Gloversville, N. Y., $4.50. The high cost at Gloversville was in part due to the fact that the filters were built covered as a protection against severe winter weather.

Operating costs of $2.40 at Columbus, $3.08 at Reading, and $1.50–$2.00 at Baltimore, are cited by Metcalf and Eddy. These authors point out that the cost of operation, other things being equal, will decrease with the increasing size of the plant; and they have estimated for certain New Jersey municipalities operating costs ranging from $2.92 per million gallons for a daily flow of 14,300,000 gallons to $5.19 per million gallons for a daily flow of 4,400,000 gallons.

CHAPTER XII

TREATMENT OF SEWAGE BY THE ACTIVATED SLUDGE PROCESS

Early American Studies on the Direct Aeration of Sewage. Various attempts, made by Waring and others, to combine filtration of sewage with aeration by the blowing in of air under pressure have been discussed in the preceding chapter. None of these efforts proved strikingly successful, and the natural aeration secured by the operation of the trickling bed was so satisfactory as to make forced aeration in combination with filtration seem, for a time, a superfluous procedure.

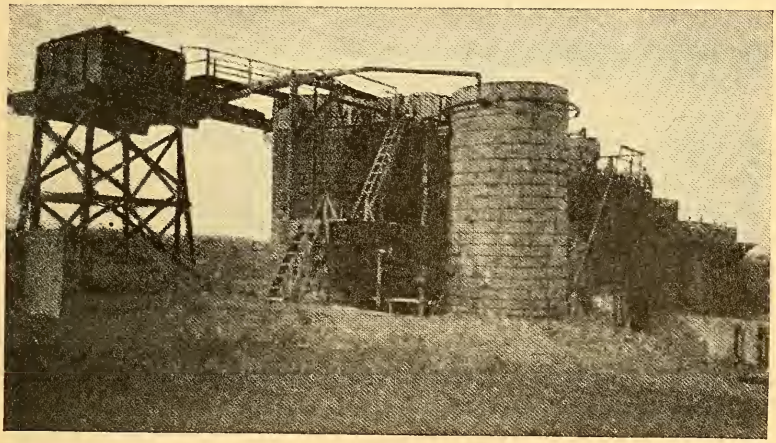


FIG. 115. Experimental Plant at Brooklyn, N. Y., showing Phelps Aerating Tank (courtesy of G. T. Hammond).

In the year 1911 Col. W. M. Black (now General Black) and Prof. E. B. Phelps presented a report on the disposal of sewage in New York harbor (Black and Phelps, 1911) which reopened the whole question of sewage aeration in a new form. The suggestion made by these authors was that the sewage in question should be subjected to a brief period of sedimentation in a septic tank and then directly aerated in a deep tank without the

use of any filtration process at all. Experiments conducted by them with New York and Boston sewages showed that the application of air in this way at a rate of 0.1 cubic foot per gallon would increase the stability of the sewage from two to threefold. These preliminary studies so impressed the city authorities that experiments on the aeration of sewage were begun in 1913. The Black and Phelps procedure involving the use of an aerating tank containing lattice-work discs placed at right angles to the entering air was studied; and other experiments were conducted with the Beddoes aerator, a device patented by C. C. E. Beddoes of Pennsylvania in 1908 for the purpose of aerating sewage and favoring its biological oxidation (Hammond, 1916).

Meanwhile experiments on aeration had been begun at the Lawrence Experiment Station of the Massachusetts State Board of Health. The first studies, made simply by aerating sewage in gallon bottles with a Richards pump, were apparently designed mainly to test the oxidizing efficiency of green algæ with which the sewage was inoculated. It was found however that aeration without inoculation with algæ yielded a considerable purification. An experiment on a larger scale was therefore undertaken in 1913. The tank used in these studies was filled with horizontal sheets of roofing slate, somewhat on the plan of the Dibdin slate bed. A ten-hour aeration period, with a supply of 200,000 cubic feet of air per million gallons, gave a reduction of 20 per cent in ammonia nitrogen, 56 per cent in total organic nitrogen, and 61 per cent in oxygen consumed (Clark and Adams, 1914).

These Lawrence experiments are of considerable historical interest because Gilbert J. Fowler of Manchester, England, to whom is due the actual discovery of the process of activated sludge treatment, visited Lawrence during the early stages of the work; and Dr. Fowler has stated that "the illuminating idea which originated the work, was due to the visit he had paid while in the United States to the Mecca of sewage purification, namely, the experiment station at Lawrence, in the State of Massachusetts, where he saw the bottle described in the paper, in which sewage had been completely purified by 24 hours' aeration" (Ardern and Lockett, 1914a, 1914b).

Nevertheless, American engineers should reciprocate Dr. Fowler's graceful acknowledgment by recognizing that none of

the investigations on this side of the water had really brought sewage aeration to a state of practical usefulness. It was the Manchester studies, and particularly the discovery of the effectiveness of previously aerated or "activated" sludge in promoting oxidation, which really gave us this important method of sewage purification.

English Experiments on the Activated Sludge Process. In Nov., 1913 Fowler and Mumford presented the first of the Manchester studies in which they reported that sewage inoculated with oxidizing bacteria was thoroughly clarified and rendered non-putrefactive by six hours of aeration. In the next year Ardern and Lockett (1914a) reported on a continuation of the same studies in which the part played in the process by aerated or "activated" sludge was first made clear. When air alone was applied directly to Manchester sewage it took five weeks of aeration to secure complete nitrification. When the sludge which had deposited during this long period was allowed to mix with fresh sewage, nitrification was much more rapid. Experiments were therefore made with larger and larger volumes of active sludge, and it was finally established that when a mixture of four volumes of sewage plus one volume of activated sludge was aerated with 15 cubic feet of compressed air per square foot of floor space for from four to six hours, the effluent after sedimentation was well clarified and non-putrescible.

The method was tested out almost at once at Salford on a somewhat larger scale. In tanks of 34,500 gallons capacity 75,000 gallons of a mixture of 75 per cent sewage and 25 per cent activated sludge were treated in 21 hours, 3 hours being allowed, in each of three cycles, for aeration, 2 hours for sedimentation, and 1 hour for emptying. A 90 per cent reduction in oxygen absorption and a 76 per cent reduction in albuminoid nitrogen was effected. The sludge contained half as much fat as ordinary sewage sludge with twice as much phosphoric acid and three times as much nitrogen (Duckworth, 1914).

The status of the activated sludge process in England in 1917 is discussed in a valuable paper by Ardern (1917). He describes a 250,000-gallon plant at Withington (Manchester), a 132,000-gallon plant at Salford, a 750,000-gallon plant at Worcester, and a 100,000-gallon plant at Stamford — all of them giving highly satisfactory results.

The engineers of the Worcester and Stamford plants, W. Jones and Jones and Attwood of Stourbridge, in a series of patents, the first of which is dated Oct. 11, 1913, lay claim to the general process of treating sewage "by aerobic action, by supplying the sewage with diffused compressed air delivered through porous plates at the bottom of a tank; so that the bacteria are continuously refreshed with air and rendered active; and the solids with which they are associated are continuously distributed and diffused throughout the liquid." These patents are abstracted in the excellent bibliography of activated sludge treatment by Porter (1917). L. C. Frank of the U. S. Public Health Service has attempted to cover this method of sewage treatment in U. S. Patent No. 1139024 issued May 11, 1915, which patent has been dedicated to the use of the Government and people of the United States without the payment of royalty. T. Chalkley Hatton of Milwaukee (Eng. News, 1916) states that the English and American claims are practically identical.

Activated Sludge Studies in the United States, 1913–1917. It is most interesting to note the way in which sewage disposal investigations in England and the United States have reacted mutually upon each other and have contributed by alternate steps to the general progress of the art. We have seen how the original studies of intermittent filtration at Lawrence led to the development in England of the contact bed and the trickling filter; and in the perfection of the activated sludge process the same helpful co-operation is well illustrated.

The 1912 experiments at Lawrence gave Dr. Fowler his germinal idea. He and his associates, Ardern and Lockett, made the really fundamental discovery of the part played by the activated sludge itself. Then Prof. Bartow of the University of Illinois brought word of the Manchester results back to America and the problem was attacked here on these new and promising lines. Prof. Bartow visited the Manchester plant and saw the work being done under Dr. Fowler's direction in August, 1914; and on Nov. 2 of the same year he began bottle experiments on aeration, with F. W. Mohlman, at the University of Illinois. In January of 1915 a small tank was put in operation, the air being supplied to the sewage through a porous "Filtros" plate, a composition of pure and carefully graded quartz fused with powdered glass. Direct aeration of sewage alone produced practically

complete nitrification in 15 days, but in the presence of previously activated sludge a similar result was obtained in 5 *hours* (see Figures 116 and 117). Later on larger experimental tanks (area 10 square feet, depth 8 feet 5 inches) were studied at the University of Illinois. A comparison of different methods of supplying air showed that one square foot of filtros plate per 10 square feet of floor area was insufficient to give satisfactory results, while 3 square feet worked admirably. The tanks operated best on a fill-and-draw plan with a 6 hours' cycle.

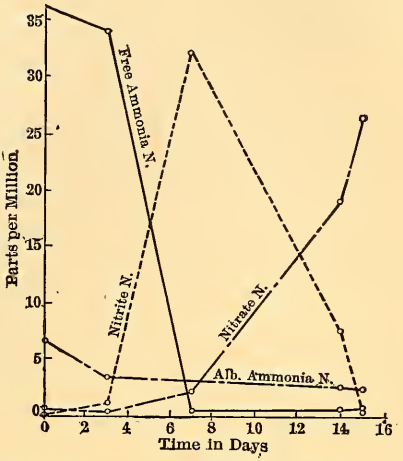


FIG. 116. Progress of Nitrification Effected by Aeration in Absence of Activated Sludge (courtesy of Edward Bartow).

The most important contribution made at Urbana was, however, the discovery that the sludge necessary for this form of treatment could be brought to the activated state in a

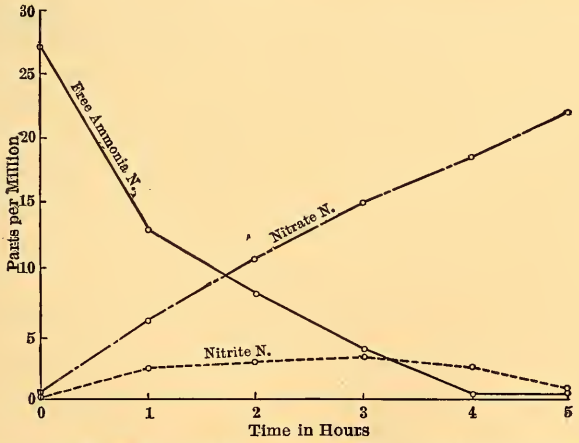


FIG. 117. Progress of Nitrification Effected by Aeration in Presence of Activated Sludge (courtesy of Edward Bartow).

much shorter time than had been deemed necessary in England. Ardern and Lockett had aerated each dose of sewage to a point

of complete nitrification but Bartow and Mohlman obtained much quicker results by a short cycle of treatment (six hours) letting the first sewage pass off incompletely nitrified but accumulating the sludge necessary for subsequent treatment at a relatively rapid rate.

Other American investigations followed without delay. L. C. Frank's experiments at the Hygienic Laboratory in Washington began in February 1915. In March of the same year the Brooklyn experimenters changed the operation of the aerating tank described above so as to make use of the activated sludge principle, while Baltimore and Milwaukee began small scale experiments on the same line. Two months later (May, 1915) investigations of the process were begun in Cleveland and at the Chicago stockyards. All of these investigations, and particularly those at Cleveland and Milwaukee which have been extended on a practical scale and for a long period of time, have contributed in an important degree to the development of the art. Among the many improvements which have resulted, the operation of the aerating tank with a continuous flow instead of on a fill-and-draw plan (patented by Frank, May 11, 1915, and first tested in a large scale at Milwaukee in June, 1915) is of special importance. For excellent acccounts of the earlier work of these experiment stations see Baker (1915) and Hammond (1916).

Principles Underlying the Process of Activated Sludge Treatment. The essential principles of activated sludge treatment may be briefly stated as follows:

The process involves the forced aeration of sewage mixed with a certain volume of previously aerated, or activated, sludge. The result of this treatment is to convert the suspended solids of the sewage into a flocculent and very readily settleable form and to oxidize more or less completely both suspended and dissolved organic matters. The effluent after the settling out of its sludge is clear and highly nitrified, and to a large degree freed from sewage bacteria. The sludge is high in water content and putrescible on standing, but is rich in nitrogen and presumably in fertilizing value. As George W. Fuller has pointed out,

"The activated sludge process has many points of similarity to the filtration of sewage in filter beds, in that it offers an opportunity for the sewage to come in contact with surfaces suitably prepared as to biological growths, promoting nitrification and other bio-chemical changes. In a sense, it is a reversal of the

filtration process in that the purifying medium is moved up and down through a tank of sewage, rather than the liquid moved through a bed of stone, each particle of which is surrounded with films containing bacterial growths and the products of bacterial growths, all of suitable physical and biological character."

The usual plant as installed in this country includes a tank or series of tanks for the aeration of the sewage-activated-sludge mixture, a separate compartment or compartments for the sedimentation of the solids and often another separate tank for the reactivation of the sludge to be added to succeeding doses.

The principal points to be determined in the design of a plant, which will be discussed in detail in succeeding sections are:

The periods of aeration and sedimentation

The proportion of activated sludge to be added to the sewage

The amount of air to be supplied and the method of application.

Optimum Period for Acration and Sedimentation of Sewage Activated Sludge Mixture. The best period for the aeration of the sewage sludge mixture is obviously closely related to the various other factors in the process, particularly to the proportion of activated sludge in the mixture and the amount of air supply, as well as to the strength of the sewage itself, and the degree of purity desired in the effluent. In the Manchester experiments it was shown that aeration for 2 hours with 40 per cent of activated sludge produced a somewhat better effluent than aeration for 4 hours with 20 per cent of sludge. The adjustment of these two factors will depend somewhat on the relative cost of aeration on the one hand and sludge storage and sludge pumping on the other.

Extensive studies of the efficiency of various aeration periods at Milwaukee gave the results tabulated below.

TABLE CII

RESULTS OF AERATION OF MILWAUKEE SEWAGE FOR VARIOUS PERIODS

(Milwaukee, 1915.)

Aeration period, hours	0	1	2	3	4	5
Cubic feet air per gal	0	0.66	1.33	1.99	2.66	3.22
Stability, hours	0	2	33	120+	120+	120+
Per cent removal of bacteria	0	52	81	92	95	98
Ammonia N, p.p.m.	22	17	15	11	7	5
Nitrite N, p.p.m.	0.08	....	0.95	1.75	2.20	2.50
Nitrate N, p.p.m.	0.08	0.04	0.70	2.80	5.60	8.20
Dissolved oxygen, p.p.m.	....	0.30	1.90	4.30	5.90	6.70
Cost per million gallons	....	$1.40	$2.82	$ 4.25	$5.64	$8.10

Even the effluent which had been aerated for only one hour was clear, showing that the physical removal of colloidal matter was practically complete; but nitrification and bacterial removal were apparent in an important degree only after three hours.

In another experiment at Milwaukee one of the tanks was operated with decreasing aeration periods and increasing rates of air supply, so adjusted that the air supply per gallon of sewage remained constant. The result as shown below made it clear that at least 3 and preferably 4 hours should be allowed for aeration even with an air supply of 1.75 cubic feet per gallon of sewage.

TABLE CIII

EFFECT OF AERATION OF MILWAUKEE SEWAGE FOR DECREASING PERIODS WITH AN INCREASING AIR SUPPLY

East Tank (Milwaukee, 1915.)

Hours of aeration.	Cubic feet of air.		Bacterial removal, per cent.	Stability, hours.	Parts per million.		
	Per minute.	Per gallon.			Ammonia N.	Nitrite.	Nitrate.
4	1.75	1.75	92.7	120	4.86	0.36	4.9
3	2.34	1.75	91.2	120	9.39	0.60	3
2.5	2.80	1.75	96.7	84	11.22	0.36	1.1

Hatton states that " In an aerating tank having 15 feet effective depth of liquor, 98 per cent removal of suspended matters can be effected with 0.5 cubic foot of air per gallon of sewage treated, applied for 1 hour, but if nitrates are to be produced in the effluent the same sewage will require from 0.75 to 1.1 cubic feet of air per gallon, applied from 4 to 6 hours " (Hatton, 1917).

With the strong sewage of the packing house district of Chicago, which contains from 21 to 260 parts per million of total organic nitrogen, 9 to 78 parts of ammonia nitrogen, 72 to 466 parts of oxygen consumed and 163 to 1040 parts of suspended solids, an aeration period of 8 hours has been recommended; while for still stronger wastes at Fort Worth a 10-hour period with 6–9 cubic feet of air per gallon of sewage may be required. (Pearse, 1918). At Cleveland it was found that 3 hours was sufficient with a mixture of 25 per cent of sludge (Pratt, 1916); while at Milwaukee 4 hours' aeration with a mixture of 20 per cent of sludge and the application of 1.75 cubic feet of air per

gallon of sewage was suggested as probably adequate (Hatton, 1916).

For sedimentation, after aeration has been carried to the desired degree, a comparatively small tank capacity is required. At Cleveland it was believed that 30 minutes sedimentation with a velocity of not over 1 linear foot per minute would suffice. Hatton (1917) states that the running through velocity should not exceed 3 feet (horizontal) per minute, and that the detention period should be from 30–50 minutes, according to the character of the sewage treated (Hatton, 1917). He points out that a vertical-horizontal flow is more efficient than either a vertical or a horizontal flow alone.

There is still difference of opinion as to the desirable depth for aerating tanks. Hatton believes that tanks 15 feet deep with a reduced amount of air under a higher pressure will prove more economical. Metcalf and Eddy however take an opposite view on the ground that irregularities of distribution are made serious with a high pressure and that in a very deep tank the bubbles of air coalesce, with a decrease in efficiency.

Proportion of Activated Sludge to be Mixed with the Sewage. This factor as pointed out above bears in general an inverse relation to the period of aeration. The Milwaukee experiments (Milwaukee, 1915) suggested a mixture of 20 per cent of activated sludge and 80 per cent of sewage. An increase of the activated sludge to 25 per cent of the mixture gave somewhat more nitrification, but the improvement did not seem to warrant the increase in construction and operating cost. At Cleveland 25 per cent was used.

It is important in determining the amount of sludge to be added to the sewage that the water content of the sludge shall be adjusted to some predetermined standard. Metcalf and Eddy (1916) point out that with a certain type of activated sludge obtained in the treatment of tannery wastes the volume of a tank occupied by sludge decreased from 25 per cent after 15 minutes sedimentation to 18.6 per cent after 1 hour, 14.7 per cent after 2 hours, and 12.8 per cent after 4 hours. The standard sedimentation period actually adopted in the United States is $\frac{1}{2}$-hour while at Manchester a 2-hour period is used.

The more concentrated the sludge which is returned to be mixed with the raw sewage, the less will be the cost of handling it,

and the smaller the required capacity of the aerating chamber for treating the mixture. At Milwaukee the sludge sedimentation basin was provided with a sludge condensation pipe at the bottom which it was hoped would reduce the moisture content to 94 per cent.

The re-aeration of the separated sludge should not be carried too far, as excessive aeration breaks up and decomposes the particles upon whose physical action the success of the process depends, and reduces the amount of nitrogen present in the sludge.

In plants where the purification of the sewage is carried to a very high degree (as at Milwaukee) special provision for reactivation of the sludge may often be unnecessary. In such a case the reactivation tank serves principally to concentrate the sludge and reduce the volume to be handled. Where the aeration of the sewage sludge mixture is only carried far enough to produce a stable effluent without complete nitrification (as in the Cleveland experiments) reactivation is essential in order to keep the sludge in good condition.

The Amount of Air Necessary and the Methods by which It can be Applied. In the experiments made at Brooklyn on the direct aeration of sewage by the Phelps-Black method it was found that as much as 18 volumes of air per volume of sewage applied for 24 hours was required in order to secure a stable effluent. In subsequent experiments, where activated sludge was mixed with the sewage 7 volumes of air with 5 hours' aeration gave better results.

The aeration of the sewage activated sludge mixture has a double function, the supply of oxygen and the agitation and intimate mingling of the organic constituents. If aeration is carried to an excessive degree it may lead to an undue disintegration of the sludge particles; and since aeration is a costly process the amount of air supplied should be adjusted as closely as possible to minimum needs in any given case.

The amount of air to be supplied could no doubt be materially reduced if more satisfactory methods of applying it to the sewage could be devised. Studies by Crawford and Bartow (1916) of the composition of the applied air and of the air leaving the sewage showed a reduction in oxygen content from 20.5 per cent to 19.3 per cent and an increase in carbon dioxide from 0.04

per cent to 0.36–0.40 per cent, indicating a consumption of about one-twentieth of the oxygen in the applied air. Nordell (1917) believes that in actual practice not more than $1\frac{1}{4}$ per cent of the oxygen supplied is utilized.

The method which has received most general favor in this country is the supply of the air through filtros plates under a pressure of 2–10 pounds per square inch. Considerable difficulty has been experienced with irregularities of distribution through these plates. Thus at Milwaukee where it was specified that a plate should pass 2 cubic feet of air per minute under a 2-inch water pressure (a 5 per cent variation either way being allowed) only 27 per cent of 780 plates complied strictly with the terms of the specification and 35 per cent of the plates received were rejected outright (Hatton, 1916). The frictional loss in passing through the plate was very great. When wet the initial loss of pressure in passing the air through under 5 pounds pressure was $\frac{3}{4}$ pound, and for every cubic foot of air per minute per square foot of surface passing the plate there was an additional loss of $\frac{1}{4}$ pound. Hatton suggests that the frictional resistance might be reduced by reducing the thickness of the plate to $\frac{1}{2}$ inch and re-enforcing it with wire or by coating the pores of the plate with paraffin (Hatton, 1916).

The filtros plates which are ordinarily 12 inches square and $1\frac{1}{2}$ inches thick are usually set at the bottom of depressions in the ridged floor of the aerating tank. The ratio of plate area to the surface area of the liquid in the tank was 1 to 3 at Urbana, 1 to 5 at Cleveland, and 1 to 8.5 at Milwaukee (Milwaukee, 1915). Later Hatton (1917) has stated that the ratio in the new tanks at Milwaukee is to be 1 to 6. He suggests that the slopes of the troughs in which the plates are set should be from 1 to 1 to 1 to 1.5 and that the plates should be placed in pitch-coated cast-iron or concrete containers, preferably the latter.

For compressing the air either positive pressure blowers or centrifugal blowers are commonly used, or when a very high pressure is desired reciprocating compressors.

The first reports from Milwaukee indicated no apprehension of clogging in the plates and as late as July, 1916, Hatton stated that with a Connersville blower specially designed to prevent the carrying over of lubricating oil and with a wood wool filter to remove dust from the air, no trouble had been experienced.

On the other hand the Third Annual Report of the Sewerage Commission for 1916 reports that the building up of the air pressure from $5\frac{1}{2}$ to 8 pounds led to an examination of the plates which showed that they were extensively stopped up with a mixture of oil and fine particles of dust. At Baltimore (Eng. News, 1916) carborundum discs stopped up badly but this was attributed to the type of compressor which was internally lubricated. At San Marcos, Texas, filtros plates clogged so that many of them had to be removed, a Connersville blower being used in this case (Elrod, 1917). The Nash blower is especially designed to supply air free from the impurities which cause such difficulties.

Alternative materials for diffusers which have been suggested instead of filtros plates are blocks of basswood and monel metal cloth. Monel metal cloth gave fairly satisfactory results in some of the earlier Milwaukee experiments and later Hatton reports that basswood, cut at right angles to the grain, gives a smaller bubble and a more even distribution than the filtros plate, with a lower frictional loss. As Bartow and Hatton have pointed out the life of the wood blocks would probably be a short one unless they were treated with a preservative and such treatment diminishes their efficiency. The most serious difficulty however is that if a tank should be emptied for any reason so as to allow the wood to dry the blocks would jump out of place when wetted again.

The most practical alternative to the use of filtros plates seems to be the application of the air from a pipe-grid through minute perforations.

G. T. Hammond at Brooklyn used pipes placed 12 inches apart with perforations $\frac{1}{16}$ inch in diameter spaced 6 inches apart, with very satisfactory results. G. L. Noble of Armour and Company reported good service from pipes of $\frac{3}{4}$-inch galvanized iron with $\frac{1}{25}$-inch holes placed 2 inches apart; but has more recently expressed a preference for filtros plates. At Moscow in the summer of 1917 there was an experimental activated sludge tank treating 135,000 gallons of sewage a day operated under the direction of Serge Stroganov. This tank was 2.5 meters deep and was operated on the fill-and-draw plan with 4 hours' aeration and 20–30 minutes' sedimentation. The air was distributed from 2.5 millimeter openings on the under side of $1\frac{3}{4}$-inch pipes

and excellent results were being obtained with the application of 5.3 cubic feet of air per gallon of very strong sewage.

On the other hand Professor Bartow in a study printed as an appendix to J. Edward Porter's bibliography of the activated sludge process reports a comparative study which indicated the

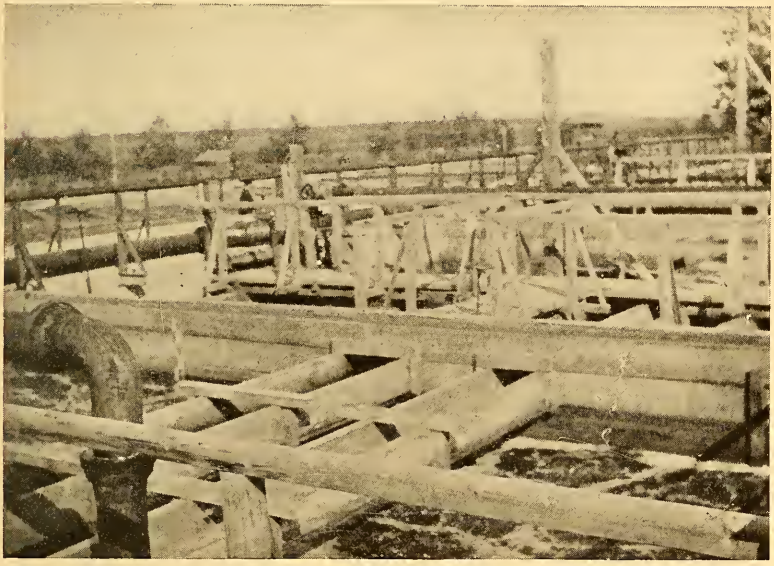


Fig. 118. Experimental Activated Sludge Plant at Moscow

superiority of filtros plates over perforated pipes with $\frac{1}{25}$-inch holes 2 inches apart, while " wood blocks were difficult to handle and even in the short time they were used showed evidence of considerable deterioration" (Porter, 1917). Hatton too while stating that " perforated pipes give good air diffusion as long as they are kept in active service and are of such material as to preclude corrosion" points out the danger of sludge being drawn into the pipes when the pressure is shut down, and concludes that " perforated pipes as air diffusers cannot be depended upon."

W. S. Coulter (1917) has suggested a process of aeration based on the entraining of air by a sewage jet and a device has been used at Hermosa Beach, Cal., for aerating sewage mechanically by the use of a diffuser wheel similar to the impeller of a centrifugal pump revolving on a vertical shaft placed at the lower

end of a sheet-iron pipe extending above the sewage level and admitting sewage at a point 1 foot below the surface. The action of the impeller produces a foam in this pipe which is forced into the bottom of the tank (Brosius, 1916). Nordell (1917) has vigorously criticised such devices on the basis of his studies at Milwaukee and believes that fine-bubble diffuser plates offer the only hope of success.

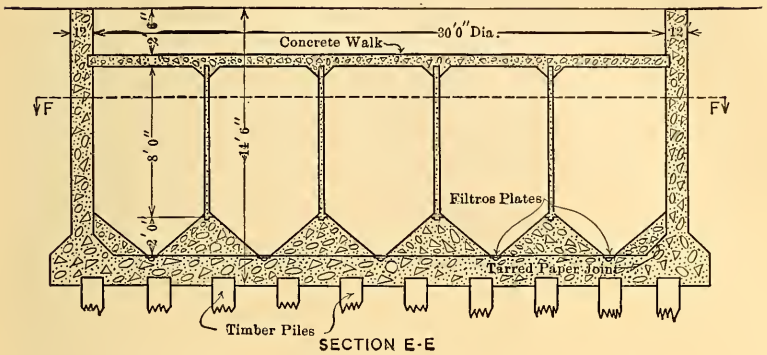
The Construction of an Activated Sludge Plant. The general principles of construction of an activated sludge plant can be best illustrated by a brief description of typical examples.

The Milwaukee plant on Jones Island, completed in 1915, as originally built included 11 circular concrete tanks, each 13 feet deep and of 30 feet internal diameter. Eight of these tanks were to be used in series as aerating tanks, 1 as a sedimentation tank, and 2 for sludge aeration.

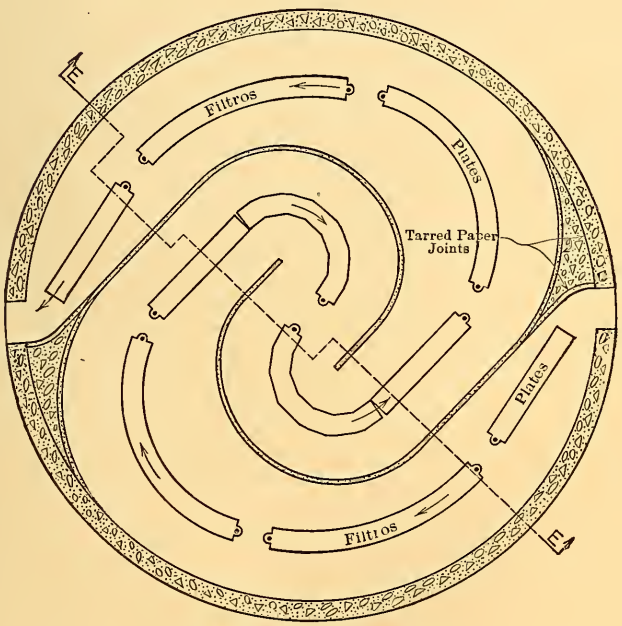
Each of the 8 aerating tanks was divided by fishhook-shaped baffle walls (see Fig. 119) so as to create a circuitous channel 6 feet wide and 114 feet long. The channel had a sloping bottom in which were set 12 by 12-inch filtros plates. The orifices supplying air to the under side of the plates were designed to pass 2 cubic feet of air per minute under 5 pounds pressure per square inch, yielding 0.25 cubic foot of air per minute per square foot of tank surface. Air was supplied by a Connersville positive blower of the Boston type, capable of compressing 2400 cubic feet of free air per minute to 5 pounds per square foot.

The tank designed for sedimentation had a hopper bottom terminating in a cast-iron pipe 4 feet in diameter extending downward 24 feet below the bottom of the tank forming a sludge condensation well. A 12-inch pipe inside the larger one extended from near the bottom of the well to the top of the tank; and inside this again was a 1-inch air pipe for forcing the sludge up to the sludge activating tank or the presses.

The ratio of air diffusing surface to water surface was 1 to 8.5. The capacity of the 8 aerating tanks was 360,000 gallons, of the sedimentation tank, 33,000 gallons, and of the 2 sludge activating tanks, 88,400 gallons, a total of 481,400 gallons. With a mixture of 25 per cent of activated sludge, a 4-hour aeration period and a 27-minute sedimentation period, 1,620,000 gallons per day could be treated. With a 3-hour aeration and 20 minutes' sedimentation, 2,160,000 gallons could be handled.



SECTION E-E



SECTIONAL PLAN F-F

FIG. 119. Design of Aerating Tank at Jones Island Plant, Milwaukee

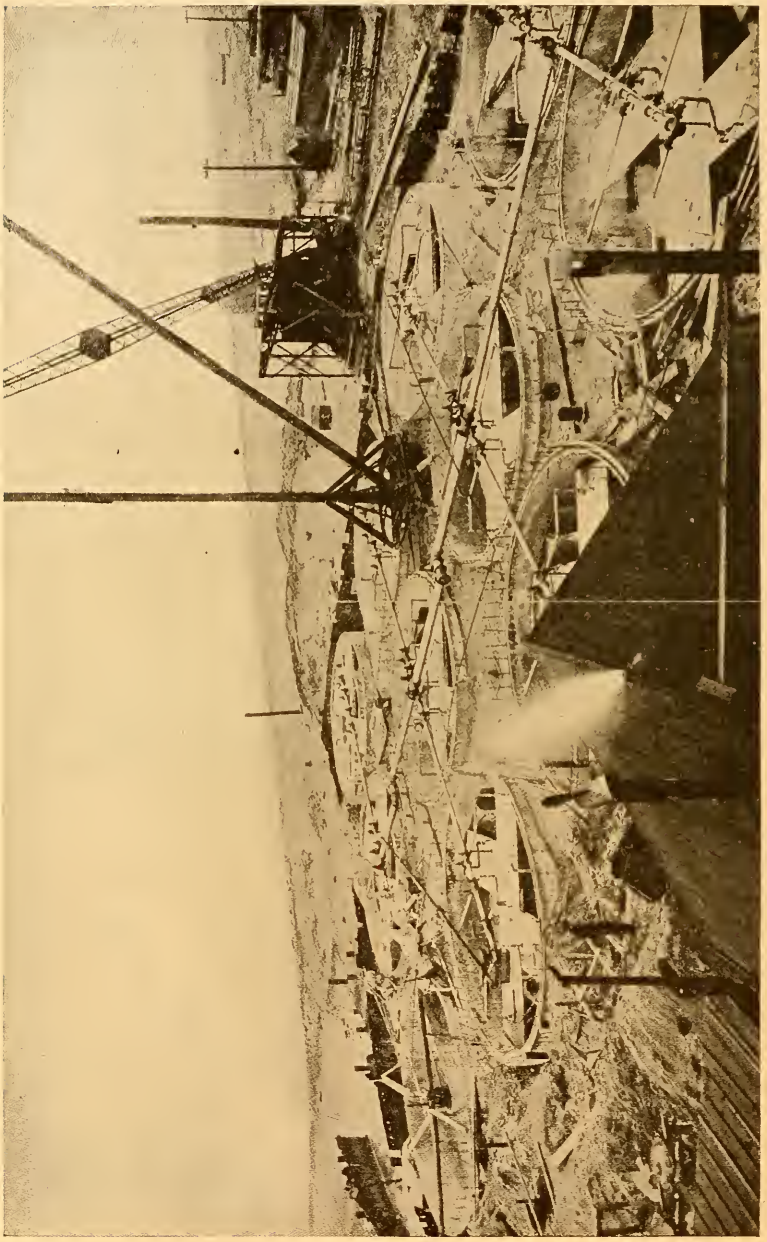


FIG. 120. Jones Island Activated Sludge Plant at Milwaukee when nearing completion.

The Report of the Sewerage Commission of Milwaukee for 1916 points out a number of important lessons learned from the first year's operation of this plant. The conduits carrying the mixture of sewage and activated sludge from one aerating tank to another accumulated sludge which became septic and used up a considerable amount of the dissolved oxygen contributed by the aeration. The filtros plates clogged badly, as pointed out above. Sludge accumulated on the 1-to-1 slopes of the hopper bottom of the sedimentation tank and became septic. The sedimentation period provided proved too short and it was concluded that a uniform horizontal velocity of 3 feet per minute or a vertical velocity of 8 inches per minute must be provided in order to secure satisfactory results.

The activated sludge plant which was placed in operation as a large-scale experimental unit at Cleveland in 1916 was highly successful and the principles of construction involved are of more general applicability than those conditioned by the plan of serial circular tanks first adopted at Milwaukee.

This plant was designed to treat 1 million gallons of sewage a day with 2 hours' aeration of the activated sludge sewage mixture, 30 minutes' sedimentation, and 2 hours of sludge re-activation. The aim of the process was to secure a moderate degree of purification at a minimum cost, complete oxidation not being called for under the local conditions. The battery of tanks (see Figs. 121 and 122) is 60 feet long and 30 feet wide, divided by two longitudinal and one sagittal cross walls into 6 chambers each $9\frac{1}{2}$ feet by 30 feet in plan. Five of these chambers are 15 feet in effective depth with bottoms arranged in valleys crossing the longitudinal direction; filtros plates at the bottom of the valleys occupying $\frac{1}{5}$ of the total superficial tank area. The sixth chamber, for sedimentation, has a hopper bottom descending to a maximum effective depth of 27 feet. In operation one of the aerating chambers was used for reactivating sludge, while each portion of the sewage sludge mixture passed through a series of two of the other aerating chambers. Sludge was removed from the bottom of the sedimentation chamber by an air-lift discharging into the sludge aerating chamber (or its inlet flume) which operated at a higher level than the other aerating chambers, so that the re-activated sludge could mix by gravity with the incoming sewage. Air was furnished by a hydro-turbine blower

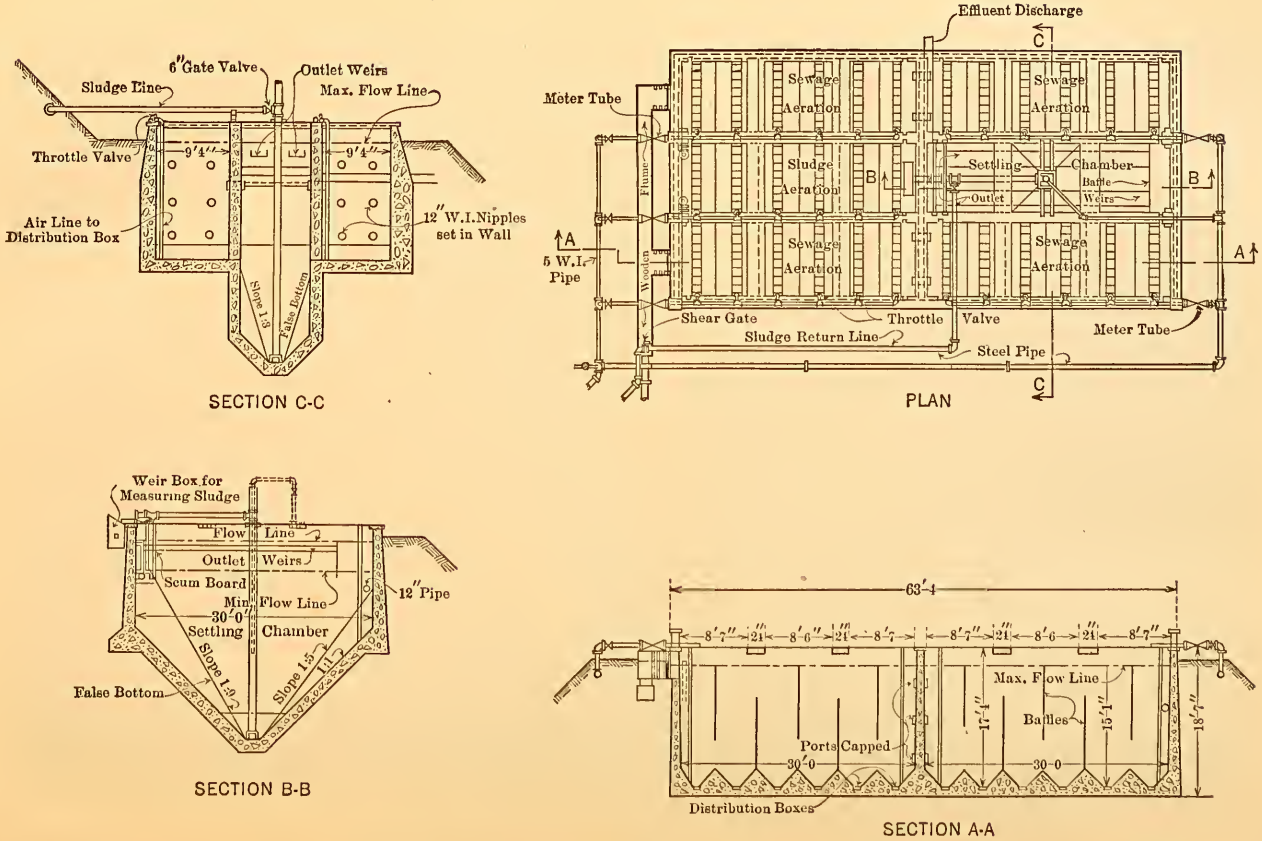


Fig. 121. Activated Sludge Plant at Cleveland, Ohio.

which delivered it to a separator and then to a pulsation-and-moisture-reducing steel tank 3 feet in diameter and 6 feet long. The compressor operated at a pressure of 9 pounds per square inch with 14.5 feet of sewage above the plates and compressed

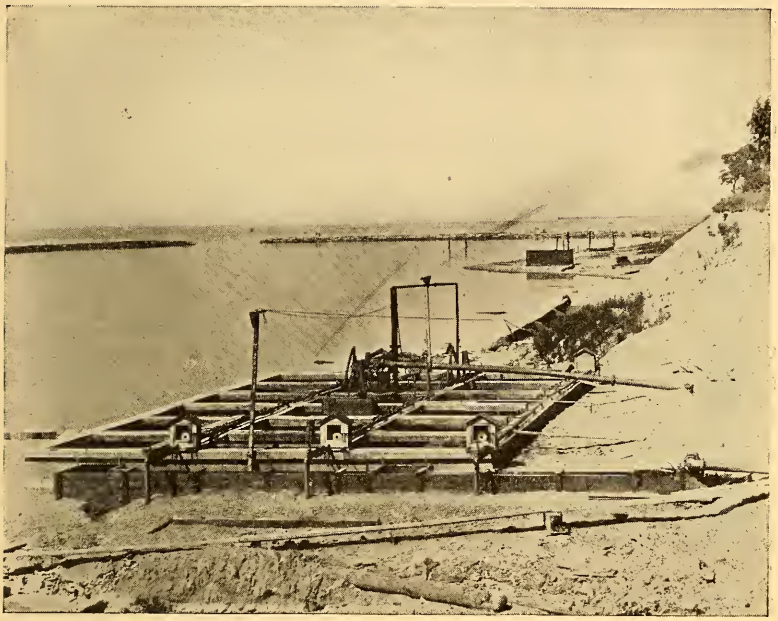


FIG. 122. General View of Activated Sludge Plant at Cleveland.

about 450 cubic feet of free air per minute. The filtros plates were set with sulphur joints in a steel box.

Activated sludge plants at Houston, Texas, with a total capacity of 19 million gallons per day, were designed along very similar plans (Williford, 1917). The plants are built up of 6 units, each having a capacity of 3,150,000 gallons per day. Each unit includes a rectangular continuous flow aeration tank (18 feet wide and 280 feet long with an effective depth of 9 feet 9 inches), a battery of 10 vertical flow sedimentation tanks and 1 sludge re-aeration tank. The ratio of aerator surface to tank surface is 1 to $7\frac{1}{2}$ and air is supplied under a pressure of $5\frac{1}{4}$ pounds per square inch at a rate of 0.26 cubic foot of free air per minute per square foot of tank surface. The aeration tanks were designed for an average detention period of 1 hour 50 min-

utes. The sludge is to be re-aerated for 3 hours and mixed with the sewage in a proportion of 30 per cent of the total. Under these conditions the air supply will be 0.33 cubic foot per gallon of sewage treated.

Results of Activated Sludge Treatment. The process of activated sludge treatment when efficiently operated should always produce a highly clarified effluent; but the degree to which oxidation of organic matter and bacterial removal are carried may be varied within a considerable range by varying the intensity and length of aeration.

The results of twelve test runs at the Cleveland plant in Table CIV show what may be accomplished in the way of moderate purification with a short aeration period and a relatively low rate of aeration.

In Table CV are shown the results obtained from the permanent plant at Milwaukee in the treatment of a much stronger sewage with a longer detention period and an air supply ranging by months from 1.76 cubic feet of air per gallon of sewage in June to 3.50 cubic feet of air per gal on of sewage in February.

In addition to the clear and highly purified effluent the activated sludge process of course yields an excess of sludge, which is very considerable in amount on account of its high water content. At Milwaukee the sludge production is estimated at over 10,000 gallons of sludge per million gallons of sewage treated, with a water content of 98 per cent. This sludge however as will be pointed out in a succeeding chapter is unusually high in fertilizing value, — containing at Milwaukee 4.5 per cent of nitrogen and 2 per cent of phosphoric acid on a dry basis. The cost of dewatering and drying the sludge and recovering its valuable manurial elements is one of the most important practical problems in connection with the general applicability of the activated sludge process.

TABLE CIV

OPERATION AND ANALYTICAL RESULTS FOR TWELVE RUNS, CLEVELAND ACTIVATED SLUDGE PLANT

Operation results — night and day sewage.

No. of run.	Quantity treated, thousand gal. per 24 hrs.	Aeration period, hours.		Settling period, min.	Free air used in cubic feet per gal. sewage treated.	Oxygen consumed.				Nitrogen.				Iron (total).		Suspended matter.		Turbidity.
						Dissolved.		Total.		Organic.		Ammonia.						
		Liquid.	Solids.			Inf.	Eff.	Inf.	Eff.	Inf.	Eff.	Inf.	Eff.	Inf.	Eff.	Inf.	Eff.	
1	472	2.2	3.3	42	1.30	32.1	14.1	63.5	17.5	9.1	4.4	11.7	11.1	.....	.....	250	38	.....
2	408	2.7	4.3	47	1.29	30.5	12.7	62.1	27.8	8	1.4	12.4	9.3	.....	.....	262	15	.....
3	429	1.3	2.9	45	1.09	30	12.3	64.5	14.1	.....	.....	11.7	8.8	.....	.....	284	....	.....
4	458	1.2	2.7	42	0.92	27.4	.....	61.4	13.5	.....	.....	13.3	10.2	.....	.....	229	.....	23
5	481	1.1	1.1	40	0.58	29.6	14.6	62.3	13.9	.....	.....	13.6	11	.....	.....	274	.....	26
6	460	1.2	1.2	41	0.47	33.4	16.6	64.6	13.4	.....	.....	14.5	12.3	.....	.....	252	23	.....
7	419	2	2	45	0.76	27.1	13.1	55.7	15.8	.....	.....	8.6	6.9	11.7	2.4	210	16	23
8	339	2.5	2.5	66	0.61	25.6	12.4	51.7	9.4	.....	.....	8.7	5.9	20.8	2.4	219	27	40
9	435	1.9	3.5	43	0.47	21.1	11.5	50	11.4	.....	....	11	7.5	20.5	2	185	19	31
10	398	2.1	3.8	48	0.62	22.8	12.7	57.2	13.7	13.3	6.8	9.2	7.2	14.2	2	255	38	38
11	383	2.3	4	55	0.68	25.8	14.4	62.4	19.2	15.4	9.5	10.7	9.8	15	6.2	309	65	98
12	282	3	3	67	0.74	26.5	14.6	60	14.8	12.5	5.6	9.6	6	21.5	1.1	234	33	35

Retention periods in activation tank based on 25 per cent displacement, runs 1–12 inclusive.

Retention periods in aerating tank based on 40 per cent displacement, runs 1–6 inclusive.

Retention periods in aerating tank based on 75 per cent displacement, runs 7–12 inclusive.

Remarks on each run. — 1, 2, 3 and 4. Good effluent. 5 and 6. Colored effluent. 7. Good effluent. Sludge not satisfactory. 8. Good effluent. Sludge darker than normal with slight odor. 9. Ebullition starts in settling tank. Poor sludge. 10. Ebullition in settling tank. Poor sludge. 11. Ebullition in settling tank very active. Sludge had bad odor. 12. Guides for controlling ebullition in place. Sludge satisfactory. Motor trouble.

TABLE CV

AVERAGE ANALYSES OF DAILY COMPOSITE SAMPLES OF SEWAGE TREATED BY CONTINUOUS FLOW ACTIVATED SLUDGE PROCESS IN THE PERMANENT PLANT AT MILWAUKEE

Date.	No. of bacteria per c.c. in millions growing at				Settleable solids, 2 hrs., cu. yds. per million gallons.		Parts per million.																	
	20°.		37°.				Suspended matter.		Total solids.		Nitrogen as										Oxygen consumed.		Dissolved oxygen.	
											Ammonia N.		Albuminoid N.		Organic nitrogen.		Nitrite.		Nitrate.					
	Inf.	Eff.	Inf.	Eff.	Inf.	Eff.	Inf.	Eff.	Inf.	Eff.	Inf.	Eff.	Inf.	Eff.	Inf.	Eff.	Inf.	Eff.	Inf.	Eff.	Inf.	Eff.	Inf.	Eff.
January..	0.761	0.190	0.172	0.028	15	1.2	336	24	1140	856	15.4	10.7	9.2	3.5	35	17	0.13	0.27	0.51	0.89	135	39	5.5	8.2
February	1.010	0.037	0.150	0.003	17.4	0.2	300	9	1260	890	17.2	19.4	10.9	2.1	42	23	0.16	0.12	0.36	0.17	143	24	4.8	6.1
March....	1.235	0.035	0.126	0.004	17.6	0.2	383	8	1250	794	15.8	14.9	11.9	2	36	19	0.13	0.25	0.41	0.41	146	24	4.7	5.6
April.....	0.768	0.045	0.070	0.002	16.3	0.1	340	3	1240	840	13.6	13.8	11.4	2.6	34	21	0.27	0.29	0.87	0.65	132	21	5.4	5.6
May.....	0.714	0.141	0.091	0.015	16.6	0.1	369	9	1270	865	13.4	12.9	10.6	2	33	16	0.24	0.06	0.64	0.42	127	24	4.4	3
June.....	1.034	0.110	0.095	0.012	15	0.7	322	17	1180	805	11.1	11.8	10.5	2.6	30	18	0.34	0.28	0.52	0.53	127	30	2.6	2.7
July.....	1.310	0.122	0.226	0.033	24	1.2	276	9	1130	800	15.6	11.1	16.1	4.4	34	18	0.14	0.46	0.05	0.32	130	32	0.5	1.5
August...	1.390	0.043	0.217	0.006	18	0.3	230	8	1064	807	14.7	6.4	19.6	3.4	35	12	0.22	0.22	0.21	1.55	117	20	0.4	2.7

Cost of Activated sludge Treatment. Actual experience has been so limited that it is difficult to estimate closely on the cost of activated sludge treatment, particularly as the gross cost of the process depends in so large a degree on the balance between the expense of sludge drying and dewatering and the value of the fats and manurial elements recovered from the sludge. The local cost of electric power for compressing air is another highly variable factor. Metcalf and Eddy point out that in small communities the cost will hardly be less than 2 cents per kilowatt-hour, making this cost item alone about $12.48 per million gallons of sewage treated.

Requardt (1917) has prepared a valuable set of curves from which the power cost per million gallons of sewage can be calculated for any given rate of air supply at any given pressure and any given power cost per kilowatt-hour. A supply of 1 cubic foot of air per gallon of sewage with a power cost of $\frac{7}{8}$ cent per kilowatt-hour would for example call for an expenditure of $3.50 per million gallons of sewage treated. The estimates are based on Connersville blowers.

Engineers in charge of the three most important plants in operation in the United States unite in placing the net operating costs somewhere in the neighborhood of $10 per million gallons.

The construction costs of the activated sludge plants at Houston, Texas, are given as $366,794 (sludge-handling equipment estimated) for a total capacity of 19 million gallons a day. This amounts to $19,305 per million gallons daily capacity or $1630 per 1000 contributing population. Williford (1917) estimates that the total cost of treatment, including interest and depreciation, will be $9.15 per million gallons.

Pratt and Gascoigne (1916) estimated that at Cleveland, power costs would amount to $3.75 per million gallons, labor costs to $0.75 and repairs, supplies and incidentals to $0.50, making a total operating charge of $5 per million gallons, exclusive of sludge handling; and they believed that fixed charges would amount to an approximately equal sum.

Milwaukee estimates placed the construction cost of the plant to be built in that city to treat 85 million gallons of sewage a day at $4,307,000 (war time prices). Interest charges on this sum will amount to $4.89 per million gallons and net operating charges, renewals and repairs are estimated at $4.75, making $9.64 in all,

a very moderate figure. The operating charges are based on the assumption that sludge pressing will cost $4.82 per dry ton, and sludge drying $3.93 per dry ton, that the sludge will be worth $12.50 per dry ton and that 1 ton of dry sludge will be recovered for each million gallons of sewage treated, so that the gross cost would be about $13 per million gallons (Hatton, 1917).

Pearse and Richardson (1917) estimate the annual cost of treating 50 million gallons a day of exceedingly strong stockyards sewage as follows: — the original construction cost for screening activated sludge treatment and sludge pressing and drying being $3,489,800, which with the addition of $261,735 for interest charges during construction makes a total first cost of $3,751,535.

Estimated annual costs for stockyards plant:

Salaries, superintendence, etc	$ 7,900
Labor	75,400
Power	270,000
Supplies and repairs	98,000
Depreciation	178,670
Interest charges at 5%	187,577
	$817,547.

This gross cost amounting to about $45 per million gallons is of course exceedingly high, but it must be remembered that the sewage to be treated is a very concentrated one. Pearse and Richardson estimate that the gross cost should be reduced by a return of $150,000 to $450,000 per year from the sale of grease and fertilizer from the sludge.

The Applicability and Usefulness of the Activated sludge Process. The experience of the past few years has made it clear that the activated sludge process fills an important place in the art of sewage treatment. On the other hand, it is by no means a panacea and whether it should be preferred to treatment on trickling beds — its nearest rival — will probably depend on various combinations of local conditions.

T. Chalkley Hatton and his associates are firmly convinced of the value of the activated sludge process as applied to the Milwaukee problem. Hatton (1917) estimates the net cost of activated sludge and trickling filter treatment as about equal and believes that when the former process develops as it should during the next decade it " will doubtless supersede the filters." He cites small land area, lower first cost and final disposition of sludge as the principal advantages of activated sludge treatment,

as well as freedom from odors and insect nuisance and small loss of head.

At Chicago, too, Pearse and Richardson (1917) strongly favor the activated sludge process for the treatment of Packingtown wastes.

On the other hand it is doubtful at the present writing (1918), whether the activated sludge process will ultimately be adopted at Cleveland; while G. W. Fuller in a careful study of the Indianapolis problem has preferred trickling filters for that city (Fuller, 1918). Fuller estimates that by making liberal allowance for the value of sludge products the cost of the two procedures would be approximately equal and the activated sludge effluent more highly purified. He prefers the trickling filter largely on the ground of the difficulty in operating an activated sludge plant, with a sewage of highly variable composition containing a large proportion of industrial wastes. At the New Haven, Conn., Experiment Station the presence of trade wastes has very materially interfered with successful activated sludge treatment. Fuller states that the sludge settled out from the trickling filter effluent in secondary humus tanks will be easier to handle than activated sludge and equally susceptible of treatment for the recovery of manurial elements.

The principal advantages of the activated sludge process are the high degree of purity attained in the effluent, the small area of land required (the treatment can be effected at a rate of about 10 million gallons per acre per day), and the freedom from local nuisance. Its disadvantages are the high power cost, the difficulties which may be expected from trades wastes and irregularities of flow with so delicately adjusted a biological process, and the large amount of very wet sludge to be handled (15–30 cubic yards per million gallons of sewage).

In such lake cities as Chicago and Cleveland, where a highly purified effluent is desired and where the land for trickling filters could only be reached at great expense for pipe lines and pumping, the activated sludge process has great advantages. Where on the other hand land for stone filters is easily accessible the problem is a very different one. Small plants where power costs are likely to be high, sludge handling relatively costly and skilled operation difficult to attain, will offer the least favorable field for activated sludge treatment.

CHAPTER XIII

THE DISPOSAL OF SEWAGE SLUDGE

Amount and Character of Sewage Sludge. Most of the processes of sewage treatment discussed in previous chapters effect a more or less complete separation of the solid and liquid constituents of sewage and produce a greater or less degree of purification of either solids or liquids or both. The liquid portion after the treatment has been completed is allowed to flow off into some adjacent body of water; but there remains in most cases a material residuum containing the solid constituents. The contact bed is practically the only process in use which does not contribute material of this kind, except when a trickling filter is used without secondary sedimentation. Even the intermittent filter contributes a relatively small amount of dry screenings and rakings.

The thick liquid suspension of residual sewage solids produced by the action of sedimentation tanks, chemical precipitation tanks, septic tanks, Imhoff tanks, secondary humus tanks following trickling filters, and activated sludge treatment is what is commonly known as sludge, and the problem of its ultimate disposal presents greater difficulties than any other question connected with sewage treatment. Sludge differs essentially from the deposit which is retained by detritus tanks, and from the solid matter deposited on the surface of intermittent filtration beds, and from screenings. As it accumulates in the tanks it is a slimy mass, containing 80–99 per cent moisture, has a specific gravity only a little greater than that of water, — 1.003–1.060, — flows by gravity, and is highly putrescible and likely to cause an aerial nuisance. The amount formed depends both on the character of the sewage and upon the process used for the preliminary treatment. With the same sewage, chemical precipitation and activated sludge treatment give the greatest amount of sludge, and the septic and Imhoff tank processes the least. With strong English and Continental sewage, using chemical treatment, 25 to 35 tons of sludge, containing 90 per

cent of water, may be formed per million gallons of sewage, while with the sewage of American cities 15 to 20 tons would be more nearly an average figure. The amount of sludge obtained from plain sedimentation is much less, being not over two-thirds of the above amounts, and, since in the septic tank process it may be considered that 20–30 per cent of the suspended matter that is deposited is liquefied or changed into gas, and since also such sludge has generally a lower water content the quantity required to be handled is about one-third of that produced by chemical precipitation.

Allen (1917) gives the following data in regard to the character of various types of sludge, based so far as dry solid content is concerned upon Worcester and Cleveland data:

TABLE CVI

CHARACTER OF SLUDGES OF DIFFERENT TYPES

(Allen, 1917.)

Process.	Volume, cu. yd. per m.g.	Moisture, per cent.	Dry matter, tons per m.g.
Imhoff	2–4	80–85	0.161
Sedimentation	4–10	88–96	0.580
Chemical precipitation	20–30	86–94	1.435
Activated sludge	20–80	98–99	0.67

Eddy and Fales (1906) give the following table of sludge yields from various processes of sewage treatment at Worcester:

TABLE CVII

QUANTITIES OF SLUDGE PRODUCED BY DIFFERENT METHODS OF TANK TREATMENT AT WORCESTER, MASS.

(Eddy and Fales, 1906.)

Process.	Tons of dry suspended matter per million gallons of sewage.			Gallons of sludge per m.g. sewage.
	Effluent.	Sludge.	Total.	
Crude sewage	1.247	.....	1.247	.....
Chemical precipitation	0.250	1.435	1.735	4872
Sedimentation	0.601	0.580	1.181	2544
Septic treatment	0.840	0.161	1.001	548

Metcalf and Eddy (1916) point out the wide variations produced in such values as those given in this table by variations in the composition of the sewage; and they cite the case of a small Massachusetts city where with a variation in sewage flow from 29.2 to 112.6 gallons per capita the sludge deposited per million gallons varied from 4271 to 908 gallons. The cubic feet of sludge per capita in this case only varied from 0.0105 to 0.0146.

The fact that so much more sludge is produced by chemical precipitation than by plain sedimentation is not due chiefly to the fact that there is a more complete removal of suspended matter, but rather to the fact that for every pound of chemicals used many times that weight is added to the sludge, one pound of dry lime forming, for instance, not one pound, but ten pounds, in a sludge containing 90 per cent water. The percentage of water that is contained in the sludge makes a much greater difference in the amount that has to be removed from the tanks than any of the other factors; and this is the reason for the large volume of activated sludge. A cubic yard of sludge containing 90 per cent water weighs approximately 1800 lbs., and if diluted so as to contain 95 per cent water, the weight would be increased to 3600 lbs.; 100 tons of sludge, having 90 per cent of water, contains 10 tons of solids and 90 tons of water, and the 10 tons of solids are sufficient to produce 200 tons of sludge containing 95 per cent of water. The reduction in weight of sludge by the withdrawal of water can be easily determined by the following formula:

$$w = \frac{S \times 100}{100 - P}$$

S = original weight of solids in 100 tons of sludge, before drying.
P = per cent of water in sludge whose weight is required (after partial drying).
w = weight of liquid sludge of specified density (after partial drying).

The following table gives the reduction in weight effected by drying 100 tons of wet sludge containing 95 per cent water:

100 tons of sludge containing 95 per cent of moisture

=	50	tons when	moisture is	reduced to	90 per cent
=	25	"	"	80	"
=	16.33	"	"	70	"
=	12.50	"	"	60	"
=	10	"	"	50	"
=	8.33	"	"	40	"
=	7.14	"	"	30	"
=	6.33	"	"	20	"
=	5.55	"	"	10	"

To determine the volume of sludge in a tank the apparatus used in the Columbus experiments (Johnson, 1905) can be employed. It consists of a glass tube about 2 ft. 6 in. long, and 0.5 in. in diameter, open at both ends. and attached to a wooden rod of sufficient length to reach the bottom of the tank. Through the glass tube there extends a fine wire, at the lower end of which is attached a rubber stopper with the smaller end uppermost, or a rubber ball. The wire is extended up to the top of the rod guided by screw eyes. In making a measurement the rod is slowly lowered through the sewage and sludge to the bottom of the tank. The stopper is then drawn, by means of the wire, into the bottom of the tube, so that when raised to the surface the tube contains a true section of the deposit.

The weight of sludge can be obtained by determining the water in the sample and assuming that a cubic yard containing 90 per cent of water weighs 1800 lbs.

TABLE CVIII

ANALYSES OF DRIED SLUDGE FROM CHEMICAL PRECIPITATION, FROM PLAIN SEDIMENTATION, AND FROM THE SEPTIC TANK AT WORCESTER, MASS.

Percentage composition of sludge.	Chemical precipitation.	Plain sedimentation.	Septic tank.
Volatile solids	47.26	51.04	43.94
Fixed solids	52.74	48.96	56.06
Silica (SiO_2)	25.46	28.59	20.41
Iron sulphide (FeS)	0.02	0.57	16.58
Iron, not as sulphide	5.80	2.45	2.98
Sulphur, not as sulphide	0.44	0.60	0.64
Aluminium oxide (Al_2O_3)	0.57	1.94	7.29
Calcium oxide (CaO)	2.88	0.61	1.14
Magnesium oxide (MgO)	0.74	0.29	0.97
Phosphorus pentoxide (P_2O_5)	0.47	1.71	1.85
Carbon (C)	28.60	31.26	23.95
Hydrogen (H)	4.21	4.46	3.64
Nitrogen (N)	2.77	3.05	3.01

The dry material in sludge contains as a rule 40–60 per cent of organic matter, and 2–6 per cent of nitrogen. The mineral constituents reported in sludges of different types at Worcester are shown in Table CVIII and the variations in nitrogen and grease content will be discussed in later sections of this chapter.

Before proceeding to a consideration of the disposal of sludge itself brief reference should be made to the allied problems of grit chamber detritus, screenings and intermittent filter-bed deposits.

Grit Tank Detritus. The solid matter deposited in detritus, or grit, tanks is quite different from sludge. It contains a much greater quantity of mineral matter, the amount of water is usually not much over 35–40 per cent, and it can easily be removed from the tanks by hand or steam shovels. The amount of solid matter deposited in the tanks depends very largely on the rate of flow, and whether or not street washings enter the sewers. Dunbar (1908) states that the total volume of the detritus is generally not more than 1.3 cubic yards daily from a population of 100,000.

At Worcester, with sewage containing more or less street washings, and with an average flow through the grit chamber of 0.5 ft. per second, about 0.12 cubic yard is deposited per million gallons of sewage. The deposit is shoveled out of the chamber, hauled about 600 feet, and covered with earth at a cost of 48 cents per cubic yard. The following data regarding the grit-chamber detritus at Worcester were furnished by Almon L. Fales:

TABLE CIX

GRIT-CHAMBER STATISTICS, WORCESTER, MASS.

Date of cleaning.	Cubic yards. removed.	Million gallons passing.	Cu. yds. per mil. gals.	Cost of removal.	Cost per cu. yd.	Cost per mil. gals.
Dec. 18, 1905.....	85	950	0.09	$35.73	0.420	0.038
Jan. 8, 1906.....	70	432	0.16	34.52	0.493	0.080
Feb. 26, 1906.....	60	506	0.12	32.46	0.541	0.064
Apr. 9, 1906.....	70	851	0.08	44.53	0.636	0.052
June 1, 1906.....	75	792	0.09	43.84	0.584	0.055
June 27, 1906.....	75	603	0.12	36.46	0.493	0.060
July 27, 1906.....	70	421	0.17	30.85	0.440	0.073
Aug. 16, 1906.....	75	334	0.22	32.98	0.440	0 099
Oct. 5, 1906.....	70	589	0.12	30.20	0.430	0.051
Nov. 3, 1906.....	80	500	0.16	31.15	0.390	0.062
Totals and averages	732	5978	0.12	$352.72	0.483	0.059

ANALYSIS OF GRIT-CHAMBER DEPOSIT

Date.	Weight per cubic yard (pounds).	Dry solids (per cent).	Loss on ignition dried sample (per cent).	Organic nitrogen in dried sample (per cent).
Dec. 18, 1905.....	1848	65.5	22	0.700
Jan. 8, 1906.....	1888	64.7	22.5	0.645
Apr. 9, 1906.....	.....	75.4	18.9	0.515
Oct. 5, 1906.....	2430	53.3	28.1	0.871
Nov. 3, 1906.....	189	61.8	21.9	0.645
Average.....	2014	64.1	22.7	0.675

Screenings. The amount and character of screenings depend largely on the size of screens used. It has been shown in Chapter III that, leaving out of consideration screens which are intended primarily to protect sewage pumps and which have therefore very wide openings, the amount removed from the sewage may vary between 3 and 40 cubic feet per million gallons.

Screenings are usually fairly solid, and contain much more organic matter than grit-chamber detritus and even more than sludge. The amount of water in ordinary screenings is not much over 75 per cent, though where very fine screens are used it may amount to 90 per cent. When the moisture does not exceed 75 per cent, screenings can be burned in a destructor, and this method is to be recommended for plants in the neighborhood of a town garbage crematory. If this method cannot be used the screenings should be buried or disposed of with the sludge.

At Cleveland it was concluded that the most feasible method of disposing of screenings from the Riensch-Wurl screen was by burning, after dewatering to about 65 per cent moisture with a small centrifuge (a ton and a quarter of wet screenings being handled daily). The screenings contained only 2.13 per cent of nitrogen on the dry basis, so that they were not thought to be of commercial value, and removal for burial at some distant point (a process recently proposed for use at Bridgeport, Conn.), was held to be undesirable on account of cost and danger of nuisance. At the 26th Ward plant in Brooklyn, N. Y., the screenings from a Riensch-Wurl installation are composted (mixed with an equal quantity of earth) and spread out on waste land.

Intermittent Filtration-Bed Deposits. The suspended matter which is deposited on intermittent filtration beds forms into more

or less of a mat and can be removed from the surface of the bed by raking or scraping (see Fig. 123). In early spring the thickness of this mat may exceed a quarter of an inch, and it can often be removed from the beds in large sheets. This mat consists largely of organic matter, paper, fat and nitrogenous substances. It is easily burned, but unless this is done in a destructor or specially constructed furnace, the odors given off are usually very objectionable. In certain places, as at Brockton, Mass., contracts have been made for the removal, free of

Fig. 123. Scraping Intermittent Filtration Beds in Spring, Worcester Sewage Works.

cost, of the total scrapings for use by farmers as a fertilizer. Where this cannot be done, the scrapings may be dug into the ground as in the case of grit-chamber detritus. They are usually only slightly putrefactive, though containing a large amount of organic matter.

Ultimate Disposal of Sludge. Some authorities have taken the view that sludge should not be formed at all, but that sewage, after passing through detritus tanks and screens, should be delivered directly on bacterial beds. This is possible, though by

no means always advisable, when bacterial purification is brought about on intermittent filtration beds, but when contact beds or percolating filters are used it is the general practice to remove suspended matter; and consequently sludge-producing tanks form an essential part of most sewage plants.

Notwithstanding this fact, much less study has been given to the subject of the ultimate disposal of sludge than to any other factor connected with sewage treatment, and how tank sludge should be handled has been more or less ignored by most engineers. Many sewage plants have been constructed, admirably adapted to produce sludge and non-putrescible effluents, but without any adequate means for dealing with the sludge, amounting, according to the method used, to from 5 to 35 tons for every million gallons of sewage.

It is possible to dispose of wet sludge at one step by dumping it at sea or by discharging it into trenches on land. As a rule, however, it is more sanitary and more economical to reduce its water content in some way by air drying, pressing, centrifugal treatment or heat drying, or by two of these methods in combination. By either of the first three of these processes the moisture content may be reduced to 75 per cent or somewhat less and by heat drying to 10 per cent. The dewatered or dried product can then be disposed of on land, or burned, or treated for the recovery of manurial elements or grease. All of these procedures must be briefly considered.

Dumping of Sludge at Sea. Carrying the sludge away in steamers and discharging it into deep water is a simple and effective procedure, and for large cities situated on the coast or on a tidal river it may be the best and cheapest method. It necessitates, however, specially constructed steamers or barges for carrying the sludge to a point where it can be discharged without danger of polluting shellfish layings, and where the solid matter cannot be carried to any foreshore. This is the method employed at London, Manchester, Salford, Dublin, at two of the plants at Glasgow, and at Providence in the United States.

At London and Manchester, according to Raikes (1908):

" The steamers used for carrying the sludge take about 1000 tons each trip, and are so arranged that when the tanks are empty the bottom is about 6 inches above the light-load line, as also is the top of the sludge when the vessel is fully loaded, and they

can thus be emptied by gravity in about 17 minutes, the discharge being regulated by means of large valves controlled from the deck, but as a rule the load is spread over a course taking an hour at a speed of about ten knots.

"The cost of these vessels is from £24,000 to £26,000 each, and in the case of London each trip involves an expense of about £15, which represents about 3½ d. per ton of sludge disposed of, in addition to the cost of precipitation and pumping into the barges."

At Providence the sludge is pressed, and conveyed to barges, which are towed out about ten miles to the United States Government Dump and then discharged. The total cost, including pressing, is quoted by Metcalf and Eddy at $2.62 per ton of solids in the sludge (moisture content, 72.4 per cent) and the cost of the sea disposal alone at 5.9 cents per cubic yard.

Disposal of Wet Sludge on Land. Sewage sludge has been successfully disposed of in certain cases by the process known as Trenching which consists of running the sludge into V-shaped trenches about 1 foot deep and 2 feet wide at the top, and covering it with earth immediately, or within one or two weeks, by turning back the ridges. The trenches should be cut one or two months before they are liable to be used, so that the earth becomes dry and disintegrated. The amount of land required is considerable, and two acres of soil of average character is not too much to allow for every 1000 tons of wet sludge, though with sandy soil one acre may suffice. At Birmingham, England, where the amount of wet sludge (94.5 per cent moisture) produced daily is about 1170 tons, three-fourths of a mile of trenches 3 ft. wide at the top and 18 inches deep is often required per day for its disposal (Watson, 1910). The same land, however, it is claimed, can be used again after a period of a year and a half or two years, if in two months or so after covering the sludge with earth the ground is broken up, planted, and, when the crop is removed, again plowed and allowed to remain fallow for about a year.

This method is comparatively free from nuisance, since the sludge is covered with earth. It is also a method of absolute disposal, as the sludge does not have to be handled a second time.

It is interfered with, however, by rain, snow or frost and during the winter months would not be a practicable procedure in cold climates.

Air Drying of Sludge. In many locations where land is not available for the ultimate disposal of the sludge by trenching there may be sufficient area for the reduction of its volume by air-drying. One method of accomplishing this end which has been adopted at several plants in England is lagooning. For the carrying out of this method, sewage lagoons are made by surrounding a given area with earth embankments high enough to contain a depth of from 24 to 48 inches of liquid sludge. The area is carefully underdrained, with drains from 10 to 20 feet apart, and is covered over with some fine material, such as cinder or gravel. When the sludge is run into these lagoons, the water is removed not merely by evaporation, but by draining, a large amount passing through the fine cinder or gravel into the underdrains.

This drainage is apt to be very obnoxious and should be treated on bacterial beds or land before being allowed to run into a watercourse. According to English authorities, the time required for drying the sludge is from two to six months. The volume of the sludge is reduced by about one-half and the water to about 75 per cent. This method of drying is cheaper than a mechanical process, but the dried product is more difficult to handle since even when in condition to be shoveled and carted away it contains more moisture than mechanically dried sludge, and there is danger of the creation of an aerial nuisance.

In this country the air drying of sludge has been practiced with considerable success, but as a rule a much less depth of sludge is treated at a given time than is the case in the English process of lagooning. The soil most suitable for the purpose is one that is open and porous, to facilitate drainage, and if not underdrained it should be plowed before using. The time required for drying the sludge so that it can be removed or worked into the soil depends greatly upon climatic conditions, for if the area is not underdrained the larger part of the drying is due to evaporation. In hot summer weather the drying may take place in a few weeks, while under other conditions many months are necessary. This method has been employed at Birmingham and at many smaller places. At Birmingham, according to Watson (R. S. C., 1908), the sludge after a few months assumes the character of rich black soil and can be plowed into the land. The amount of land required is from one to two acres (depending on the character of

the soil) per 1000 tons of sludge. Fuller (1912) recommends 350 square feet of bed per 1000 population for septic or Imhoff sludge from a separate system of sewers, to be increased by 50–100 per cent for combined systems or sewage containing trade waste.

Careful studies were made at Worcester on the air drying of Imhoff sludge and of the sludge obtained by settling trickling filter effluent in humus tanks. Both were handled very successfully provided the weather was warm and dry. A depth of 12 inches of sludge could be handled in hot summer weather as easily as 6 inches in spring or fall. The following description of the process is from the report of Matthew Gault for 1912 (Worcester, 1913).

" When first drawn onto the drying bed, the Imhoff tank sludge became completely covered with gas bubbles, due to the release of pressure on drawing from the tank. The escaping gas lifted the solids, leaving a comparatively thin liquid at the bottom just as in the case of the glass cylinders where inside of 24 hours, from 15 to 30 per cent of clear liquid remained at the bottom. This clear liquid filtered away within a day or two, reducing the volume of the sludge correspondingly.

" Under favorable weather conditions, the surface of the sludge on the second day presented a thin unbroken crust. When the sludge was stirred up, however, much gas escaped, making a sound like a man puffing on his pipe, and within a few minutes the surface where it was broken looked as if it were strewn with blackberries, due to the gas bubbles and the sludge. The entire mass where disturbed, except for color, now resembled soft dough in which yeast is very active. Within a few hours after this condition, cracks began to appear on the surface and gas escaped through these cracks, causing the sludge to become a seething mass. The evolution of gas soon subsided, although the sludge retained much gas for several days, rendering the sludge porous and spongy. The cracks gradually increased in number and deepened, thereby facilitating the drying. The bottom portion was the last to dry, the time required to dry out depending upon the depth of sludge.

" The dried sludge was very porous and light in weight. It contained much hair, which came largely from the tannery and which is very resistant to the process of decomposition. This hair acted as a binder so that the sludge could be removed in large

pieces. (See Fig. 124.) No foul odors developed during the drying, and the odor of partially dried sludge was merely somewhat musty and tarry. When it had become well dried out, it was readily ignited with a burning wisp of hay and burned with a blueish white smoke without offensive odor. When it had developed considerable heat, it burst into flame.

"Under the most favorable conditions of air-drying, the moisture content was reduced as low as 10 per cent."

Air drying on open beds at Cleveland (1914) reduced the moisture content of Imhoff sludge from an original value of 84–89

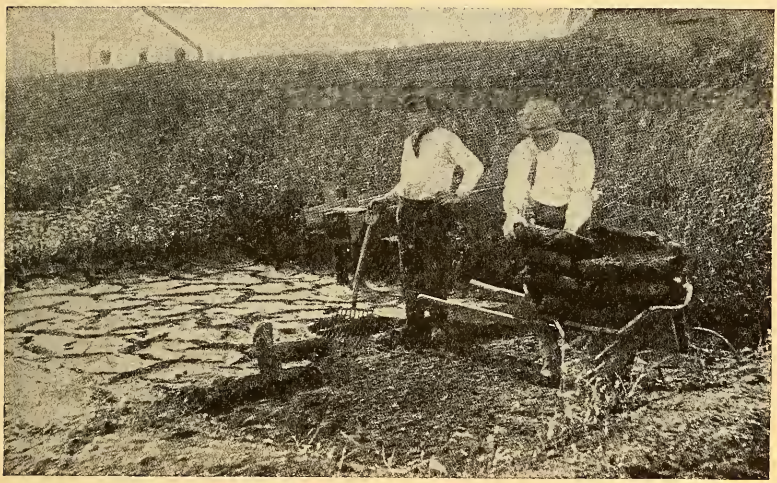


Fig. 124. Removing Air Dried Imhoff Sludge from Drying Bed at Worcester (courtesy of Matthew Gault).

per cent to 69–76 per cent after 3 days and to 48–71 per cent after 7–8 days. The less satisfactory results were due to unfavorable weather conditions; and these experiments led to the use of drying beds roofed with glass after the general plan of a greenhouse. The first of these beds to be constructed was 10 feet by 20 feet in plan and built with a tight wooden bottom sloping to a central drain. Over the floor was laid 5 inches of 1–2-inch limestone, then 4 inches of 1-inch limestone, 3 inches of gravel and 1 inch of lake sand. To protect the sand a layer of 1–3 inches of dry grit from the detritus chamber was spread over the surface before dosing the bed with sludge, the grit being removed each

time with the dried sludge. Imhoff sludge containing 85–93 per cent of moisture was successfully dried at all seasons on the covered bed at Cleveland, and it was estimated that while an open bed could only be relied upon to treat 13 doses of sludge (during the warmer months) a covered bed should handle 32 doses a year. With an average depth of 10 inches per dose of sludge of 87 per cent water content one square foot of a covered bed should dry to a spadeable condition one cubic yard of sludge per year. This computation assumes that the covered beds will

Fig. 125. Glass-covered Drying Beds at Alliance, Ohio.

be kept heated above the freezing point during the three cold months of the year.

A patent upon this process of drying sewage sludge under glass has been granted to one of the authors (R. W. P.), but it will be used only for defensive purposes and not for the collection of royalties. Glass-covered beds have been constructed at Canton and Alliance, Ohio, the Alliance beds which cover a quarter of an acre and cost $8700 complete (in Jan., 1917) being shown in Fig. 125. The Canton bed has been in operation for two winters and there is no doubt that it did three times as much work per square foot during the year as could be done on an open bed. Glass-covered beds have the advantage of permitting one force of attendants to work all the year round and hence promote

efficiency. Even in the summer time glass covers are a protection against the wetting of nearly dried sludge by rain storms.

Dewatering of Sludge by the Use of Sludge Presses. Though air-dried sludge can be spaded, the drying requires, as has been noted, a large area of land, and is very liable to create a nuisance; so for large works the reduction of moisture must as a rule be brought about by mechanical means. The method commonly used is known as sludge pressing, although recently plants have been built for removing the moisture by centrifugal force.

Sludge pressing reduces the sludge to about one-fifth of its original volume, and the amount of water to 60–70 per cent. The mass thus obtained is much less putrescible than wet sludge, has much less odor and can be shoveled and easily carted away from the proximity of the works. Pressing avoids to a large extent the nuisances that arise from land treatment of fresh sludge, and renders sludge treatment possible at many places where formerly every foot of available ground had been taken for air drying or sludge lagoons.

By this process the wet sludge is pumped under heavy pressure between sheets of cloth or canvas, which retain the solid matter, but allow the water to be forced through them and out of the chambers of the press. The sludge remains between each pair of cloths in a cake from one to two inches in thickness.

There are numerous forms of sludge presses, although they all consist essentially of a series of cast-iron plates. The plates are of various shapes, but usually have an area of about one square yard and are mounted on rollers upon side rods which hold the head and tail of the press together. The number of chambers varies from fifty or less to one hundred and twenty-five or more per press. The plates are usually concave on each side and are corrugated. The rims are faced so as to make tight joints when the plates are in contact with each other, thus forming chambers from one to two inches wide, in which the sludge is collected. Each plate has a hole about six inches in diameter in its center. On both faces of each plate are hung cloths which are either sewed together around the central hole in the plate or clamped to the plate at this point. When in use, sludge is admitted under pressure through the central channel and passes out into the bags, the water squeezing through and passing off by the corrugated channels mentioned above.

When no more liquid can be forced out at the pressure used, the plates are separated and the sludge cakes removed from the canvas.

Fig. 126, reproduced by courtesy of the John Johnson Co., shows one form of sludge press commonly used.

The general method for pressing sludge is to run the sludge from the sedimentation or septic tanks into a sludge well, add lime in the form of milk of lime, allow the sludge to settle for 12 to 24 hours, draw off the supernatant liquid, and force the sludge by means of compressed air through bar screens into the sludge presses. The addition of lime used to be considered necessary in order to facilitate the pressing of slimy sludge. Lime amounting

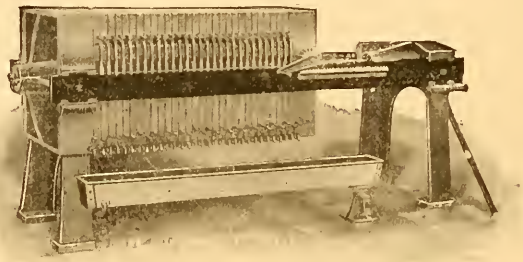


Fig. 126. Sludge Press, for pressing Sewage Sludge (courtesy of John Johnson Co.).

to from 3 to 7 per cent of the weight of the pressed sludge was added, the smallest amount being required for sludge from chemical treatment, the most for septic tank sludge. Where nitrogen is to be recovered, however, the addition of lime is highly undesirable, since by its chemical action it decomposes a considerable part of the nitrogen present. The pressure used for pressing sludge with lime is usually varied from 60 to 70 pounds per square inch and the time required from a quarter to a half an hour. Where activated sludge is treated for the recovery of nitrogen lime is not used, but instead the pressure is increased to 125 pounds per square inch and the time is prolonged to 1 or 2 hours.

The liquid pressed from the sludge contains a large amount of putrescible matter, is strongly alkaline and gives off a strong odor of ammonia. At plants where chemical precipitation is

used it can be run back into the sedimentation tanks, thus reducing to a certain extent the amount of lime required. At other places it can be added to the sewage or treated on fine-grained bacteria beds.

The careful operating records kept at Worcester (Worcester, 1916) give the following data in regard to the pressing of chemical precipitation sludge: Average annual values from 1899 to 1916 for water content of sludge range from 97.01 per cent to 89.78 per cent; for water content of pressed cake, from 73.9 per cent to 67.8 per cent; tons of solids per million gallons of sewage treated, from 0.94 to 1.98; operating cost per million gallons, from $3.85 to $7.05; operating cost per ton of solids, from $3.39 to $5.71.

The Johnson type of press described above is the one which has been most generally used in the past for the pressing of sludge. A number of other types such as the Kelly press, the Sweetland press, the Worthington or Berrigan press and the Simplex Ejector press have been tested with promising results in recent years, particularly in connection with the dewatering of activated sludge. Allen (1917) gives good descriptions of all but the last of these forms. At Cleveland the Sweetland press reduced Imhoff sludge from 86 per cent to 62–76 per cent moisture; and at Milwaukee the Berrigan press, treating activated sludge containing 96–97 per cent moisture, produced a cake containing 75 per cent of moisture. Both the Cleveland and Milwaukee studies were conducted without the addition of lime.

Sludge Drying by Centrifugals. This method of removing water from sewage sludge is of a recent date, the first plant having been erected at Harburg-on-Elbe at the close of the year 1907.

The sludge that is obtained by this process is said often to contain only 50 per cent of water, the amount varying from 50 to 70 per cent; and at Harburg it was not necessary to add lime to the sludge before running it into the centrifugal.

The cost of the drying is given as ranging from 5 to 7 cents per cubic yard of sludge treated. According to *Engineering* (1909): "The machine consists essentially of a drum with a vertical hollow shaft, revolving at a high speed. This drum is fitted with six radial chambers arranged in a star form, each chamber being divided into two sections by a sieve down the

center; one of these sections has parallel sides, and is fitted with valves at the inner and outer ends, these valves being operated by oil under pressure, while the other is of a radial form and contains a drainpipe communicating with an annular canal situated below the drum.

"The method of operation is, briefly, as follows: With the drum revolving, the inner valves open and the outer ones closed, the feed-valve from the tank above is opened and sludge is introduced into the parallel sections of the chambers by way of the hollow shaft and the inner valve-ports. The solid matter present is then immediately carried, by centrifugal force, to the outer part of the chambers, the lighter water being forced back, and thus separated from the solids; this separated water passes off through the sieves into the drainpipes and out of the machine by way of the annular canal. As the process is continuous, the parallel part of the chambers finally becomes filled with a dried mass, the whole of the separated water passing off by the overflow. The inner valves are now closed, and the outer ones opened, when the dried mass is projected outwards against the casing of the machine, striking which, it is broken into small pieces and falls, by way of a funnel, into a receptacle or conveyer below. In the act of being projected outwards, the dried sludge slides past the sieves, and thus frees them from small adhering particles which might tend to close up the meshes and impede the free passage of the water." (Fig. 127.)

At Harburg the fresh sludge, not more than two days old, is run from the sedimentation tanks through $\frac{5}{16}$-inch bar screens and forced into a cistern provided with mechanically operated stirrers and mounted over two of the centrifugal machines. The sludge, in as uniform a condition as possible, is delivered to the different chambers of the centrifugal through a pipe in the center of the revolving chamber. The solid particles of the sludge are thrown against the outer part of the compartments and the water, which is lighter, is held back and runs through the strainers to the outlet channel and back into the sedimentation basin. The space occupied by the water is filled with sludge from the settling tank, and in a very short time all the chambers are filled with a dried sludge, which is then thrown out upon the conveyer.

Two of the centrifugals have been continuously in operation at Harburg since 1907, treating from 550 to 700 cubic feet of the

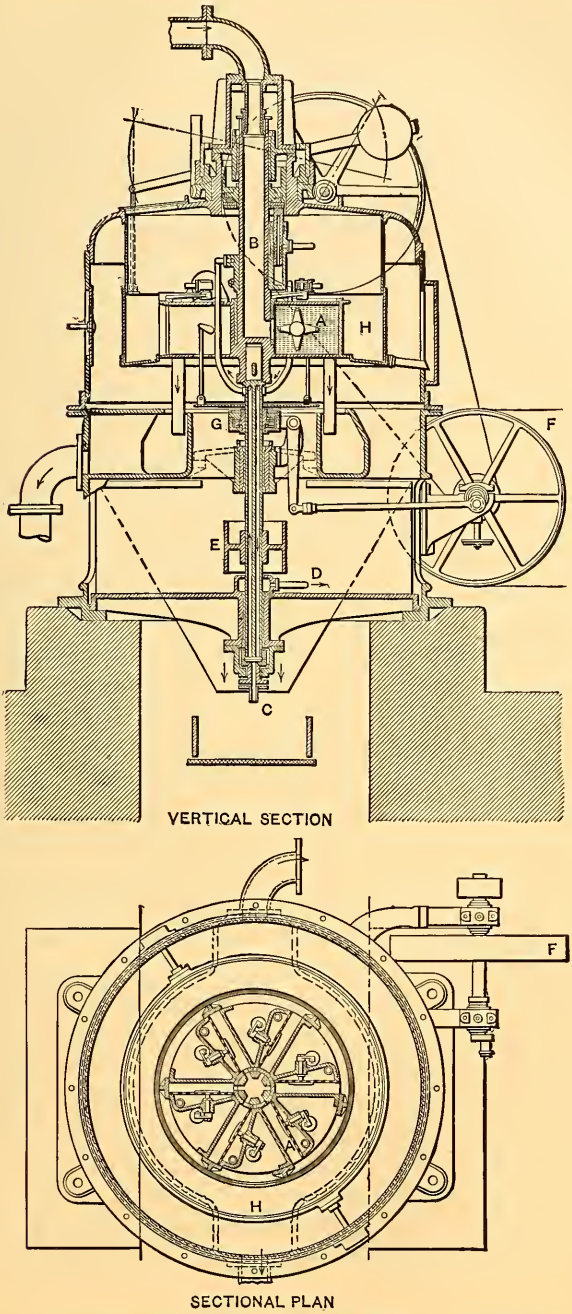


Fig. 127. Centrifugal Dryer for Sewage Sludge (Engineering, 1909).

sludge per day. The machines are electrically driven, and the energy required to drive the plant is 100 kw. per day. At Frankfort-on-the-Main an installation of six machines was erected, to be run night and day and to handle from 260 to 325 cubic yards of sludge in 24 hours. The dried sludge was to be burned in a destructor in connection with the town refuse.

A possible advantage of centrifugal treatment in certain cases may lie in the fact that the grease in the sludge will go off with the liquid, thus freeing the sludge to be used for fertilizing purposes from an undesirable constituent. On the other hand it should be borne in mind that the liquid produced by centrifugal treatment is far more impure than that produced by sludge pressing and careful provision should be made for its disposal. At the Cleveland Testing Station experiments were made in a small way in dewatering activated sludge on this plan and negotiations have been started to secure the co-operation of the centrifugal machine manufacturers to make some thorough tests.

The Dickson Process of Dewatering Sludge. An interesting method of dewatering sludge on an entirely different plan was first introduced by Dickson at Dublin, Ireland. The operation of an experimental plant operated with plain sedimentation sludge at Brooklyn is described as follows by Fuller (1916): "The sewage is subjected to plain sedimentation in hopper-bottom tanks from which the sludge is removed at intervals and a portion of the water removed by further sedimentation. This sludge, with 92 to 96 per cent of water, is mixed with about $\frac{1}{2}$ per cent of brewers' yeast and allowed to ferment for about 24 hours, in long, narrow, round-bottom, sloping tanks, which are jacketed and kept heated to about 90°. Anaerobic bacteria decompose the yeast cells and cause the sludge to become so mixed with entrained gases that the solid matters rise to the top in a manner similar to that noted with Imhoff tank sludge, and a fairly clear subnatant water is removed through a perforated pipe at the bottom of the tank. The water content of the remaining sludge averaged about 80 per cent, according to analyses made by Hill and Ferguson, but some samples showed 75 per cent, which, with improved devices could probably be obtained regularly."

A somewhat similar principle is involved in the procedure tested at Cleveland and patented by one of the authors (R. W. P.) for causing sludge to rise to the surface of a tank for removal in

a somewhat dewatered condition by introducing heat at the bottom of the tank. This procedure proved entirely feasible but is probably not to be recommended for use on a practical scale.

Heat Drying of Sewage Sludge. The product of air drying, pressing or centrifugal treatment is a solid material, usually containing 70–75 per cent of moisture, which has been reduced to a fraction of the volume of the wet sludge and can be easily spaded and transported. If fats or manurial elements are to be recovered it will be necessary to carry the process of drying still further so as to reduce the moisture content to approximately 10 per cent.

This is generally accomplished by the use of heat dryers of some form. In a test made at Milwaukee with a Smith type of indirect heat steam-jacketed dryer 3200 pounds of press cake containing 71.7 per cent of moisture were reduced to 789 pounds of dried sludge containing 10.5 per cent of moisture. 2880 pounds of steam were used in drying and 64 pounds in maintaining pressures; 44 amperes of electricity and 209 volts were required and the time consumed in drying was $7\frac{1}{3}$ hours.

Other estimates of the cost of drying sewage sludge will be cited in a later section.

Land Disposal of Dry Sludge. Dry sludge can be easily disposed of by digging it or plowing it into the land. This can be done as soon as the sludge is delivered from the presses or centrifugals, and the area required is not over one-fifth that necessary for trenching wet sludge, as the volume of the sludge is reduced four-fifths during drying. Probably the area required is much less than the above figure, as the same land can be used more frequently than when wet sludge is applied. Winter weather again does not interfere seriously with the land treatment of dry sludge, as sludge containing 60–70 per cent of moisture is not very quickly putrescible.

When there is not sufficient land available in the immediate vicinity of the plant for digging or plowing in of the sludge, or where the farmers cannot be relied upon to remove it, the sludge must be carried away to some isolated locality or burned. Dry sludge can also be used as a filling material if the locality selected is sufficiently distant from traveled roads and dwellings, and there is no probability of the land being used for building purposes.

Where this method is possible, it is, next to sea disposal, the most satisfactory way of treating sludge.

At Worcester, Mass., the sludge, when loosened from the cloths of the sludge presses, drops into conveyers, running under and parallel to the presses. The conveyers consist of iron troughs in which 6-inch iron lugs attached to endless wire cables travel in one direction and drag the sludge through the troughs and drop it into cars at the outer end. The sludge cars are hauled by an

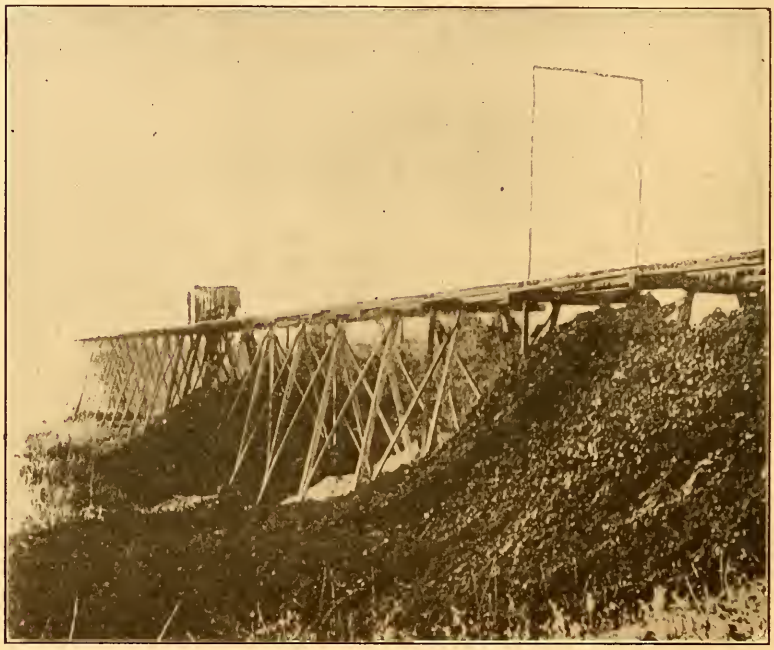


Fig. 128. Dumping Pressed Sludge, Worcester Sewage Works.

electric motor car to a deep isolated valley three-quarters of a mile from the press house, and the sludge is dumped from a trestle, the location of this trestle being changed as the occasion demands. Such favorable conditions as these for the disposal of the dried sludge are somewhat unusual (Fig. 128).

Burning of Sludge. Many experiments on a large scale have been made in regard to the burning of sludge, and it has been shown that there is no inherent difficulty in burning sludge when the moisture content is not over 60 per cent, and that sludge

containing 75 per cent of water can be burned with careful manipulation.

At Worcester, Mass. (Eng. News, 1892*a*), in 1891 several experiments were made on the burning of sludge in a furnace with low chimney, both with and without forced draught. In one of these experiments, 45 tons of sludge, containing about 46 per cent moisture, were burned, three cords of wood being used as fuel. The clinker formed was a semi-fused mass, due to the large amount of iron contained, and was not difficult to remove from the furnace. The cost, chiefly for manual labor in collecting the sludge from the sludge beds and bringing it to the furnace, amounted to $3 per ton of dry sludge. In another experiment sludge containing about 72 per cent moisture was burned at the rate of 2.2 tons in 9 hours, aided only by a very small amount of fuel.

In England, at Ealing-on-Thames, sludge has been successfully disposed of by burning. Ealing is about 9 miles from London, and has a population of about 35,000. The sewage is treated with lime, clay and aluminium sulphate. In 1883, by the advice and under the supervision of Charles Jones, a destructor for the burning of the town refuse and sludge was built. The destructor plant consisted of seven cells of Fryer's patent destructors with a grate area of 5 by 5 feet each, to which, in 1889, was added a fume crematory, which consisted essentially of a chamber in which a fresh coke fire was maintained, over which all the gases arising from the fires in the destructors were passed before allowing them to escape into the air.

The sludge was at first mixed in the wet state with the town refuse and later drained. In 1902 this method was given up and the sludge was pressed before burning. The pressed cake, containing about 60 per cent of water, was conveyed by a hydraulic lift to the destructor furnaces and then mixed with 1.5 to 2 parts dry house refuse to 1 part sludge cake. The steam generated was utilized for various purposes at the plant. From 2500 to 3000 tons (R. S. C., 1908) of pressed sludge were thus disposed of per annum.

Jones states that the cost of burning generally works out at from 36 to 40 cents per long ton, but he believes that there is no real cost, but rather the reverse, as the clinker that is obtained is used for various purposes, as walling, concrete, bacteria beds, etc.

Extensive experiments have also been carried on by Campbell at Huddersfield (R. S. C., 1908). The combustion plant there consisted of a two-cell Horsfall destructor, each cell having a grate area of 30 square feet with a drying hearth at the back. Each cell was fitted with a steam blower. Campbell's original intention was to mix the pressed sludge with an equal amount of town refuse, but on account of the difficulty in delivering the town refuse at the sewage works, coke breeze was used, and for part of the time gas-works clinker, containing a fair amount of coke and combustible matter, was substituted. The pressed sludge was tipped into feeding bins at the back of the furnace and there mixed with the coke breeze or gas-works clinker in the proportion of one part fuel to five parts sludge cake. The fumes given off from the pressed sludge in drying passed over the hottest part of the fire in the destructors before they escaped to the chimney. The average cost of burning the pressed sludge (R. S. C., 1908) was 63 cents per long ton.

Recovery of Nitrogen and Other Manurial Elements from Sludge. The use of sludge or some of its constituents for fertilizing purposes has always had strong advocates on account of the belief that use should be made of the nitrogen, phosphoric acid and potassium salts contained in the sludge. Sludge undoubtedly has a manurial value, but the amount of manurial constituents is relatively very small, and consequently its value to the farmer as a fertilizer depends to a large extent upon the cost of carriage. At Canton and Alliance, Ohio, certain farmers have been hauling sludge from the sewage plants at their own expense for years past, although the practice has not become general.

The results obtained by the Royal Commission in their fertilizing experiments, after one year's study, suggest not only that the manurial constituents of sludge act much more slowly than those of ordinary artificial manures, but also that, unit for unit, the nitrogen and phosphoric acid are of less value than in artificial fertilizers. There is, moreover, the danger of clogging up the soil if too large an amount of sludge is used, especially if the sludge is from chemical precipitation, on account of the large amount of iron and lime salts and grease it contains. Consequently, to find a market for dried sludge is, and always has been, difficult, and a city may consider itself fortunate if the

removal of the dried sludge from the neighborhood of the sewage plant can be accomplished free of expense. Two plants, however, in Great Britain seem to have been more or less successful in marketing their sludge. One is at Kingston-on-Thames, and the other the Dalmarnock plant at Glasgow.

At Kingston the chemicals used as precipitants are alumino-ferric, blood, charcoal and clay. The sludge after removal from the tank is pressed, partially dried by heat, and sieved, and after further air drying during storage is sold under the name of "Native Guano." The analysis of this product, as given in the Fifth Report of the Royal Sewage Commission, is as follows:

TABLE CX

COMPOSITION OF NATIVE GUANO

	Per cent.
Moisture (at about 110° C.)	25.87
Matter volatile on ignition	37.99
Non-volatile matter	36.14
	100.00

ANALYSIS OF NON-VOLATILE MATTER

	Per cent in the sludge.
Grit (*i.e.*, matter insoluble in hydrochloric acid, after ignition)	22.33
Oxides of iron and alumina	10.10
Lime	3.30
Potash (soluble in dilute hydrochloric acid) approx	0.16
Potash (soluble in water)	0.06
Phosphoric acid (P_2O_5)	1.74
Equivalent to tribasic phosphate of lime	3.30

Yield of nitrogen (total)	1.93
Nitrogen evolved as ammonia on boiling for 2 hours with a dilute solution of potash (0.5 per cent KOH)	0.41

At the Dalmarnock plant at Glasgow the precipitants are lime and ferrous sulphate. The sludge is pressed and dried at a temperature of 165–170° F., passed through a pan mill and marketed under the name of "Globe Fertilizer" at $1.95 to $2.40 per ton in bulk, or $3.40 per ton in bags.

The results of the analyses of this fertilizer, as given by the Royal Commission, are as follows:

TABLE CXI

COMPOSITION OF GLOBE FERTILIZER

	Per cent.
Moisture (at about 110° C.)	22.51
Matter volatile on heating	33.98
Non-volatile matter	43.51
	100.00

ANALYSIS OF NON-VOLATILE MATTER

	Per cent in fertilizer.
Grit, etc. (*i.e.*, matter insoluble in hydrochloric acid, after ignition)	10.75
Oxides of iron and alumina	13.42
Lime	12.09
Potash (soluble in hydrochloric acid) approx.	0.10
Phosphoric acid (P_2O_5)	1.11

Equivalent to tribasic phosphate of lime	2.42
Yield of nitrogen (total)	1.30
Nitrogen evolved on boiling the sludge for 2 hours with dilute (0.5 per cent) potash solution	0.06

A large portion of the pressed sludge is not artificially dried, being sold as pressed cake at 16 to 25 cents per ton in bulk. Though all the sludge from the Dalmarnock outfall is disposed of to farmers, it is to be noted that sludge produced at the Dalmuir and Shieldhall outfalls is carried out to sea.

Clark has estimated that the nitrogen in sewage would theoretically be worth $52 per million gallons, the fats, $7, the potash, $5, and the phosphorus, $4; but a large proportion of the materials are not present in an available form. Much of the phosphoric acid for instance is insoluble and therefore useless. The water and grease content of the sludge is the great obstacle to its use. The Metropolitan Sewage Commission of New York (1914) concluded that a sludge containing 50 per cent of moisture on the wet basis and less than 10 per cent of grease and at least 3 per cent of ammonia on the dry basis might be further dried and ground at a profit on a large scale, but that "no other than an occasional and uncertain offset to a part of the cost of operation can be looked for, even under favorable circumstances, from the sale of sludge in the form of crude cake or containing over 30 or 35 per cent of moisture."

In spite of these inherent difficulties, there has been a marked reawakening of interest in the possibility of recovering valuable

elements from sludge in recent years, largely as a result of the fact that two of the new modes of treatment, the activated sludge and the Miles acid processes, yield sludges of much higher content, in nitrogen and grease respectively, than do any of the earlier procedures.

The manurial value of sludge is fixed according to state laws by the result of the alkaline permanganate method as used in the New England States and by the results of the neutral permanganate method in the South Eastern States. The true manurial value cannot however be determined by such empirical analysis since the availability of the nitrogen varies widely; and according to Lipman and Burgess (1915) the same sludge may yield quite different results with different soils. These investigators tested the value of different fertilizing materials by mixing them with moist soil, incubating for a month and determining the nitrate nitrogen formed in the soil by bacterial action. The percentage of nitrate nitrogen made available in this way was very much higher in the case of septic and Imhoff and activated sludge than in the case of ordinary low-grade fertilizers, and was comparable with that characteristic of such high fertilizers as dried blood and high-grade tankage. These results appear to be in conflict with the findings of the British Sewage Commission cited above. So far as activated sludge is concerned (this type of sludge was of course not studied by the English investigators) the Lipman findings are confirmed by Bartow (1916). As a result of studies of the effect of activated sludge and other materials upon plant growth he concludes that this material contains nitrogen in an unusually available form or that the phosphorus which it contains greatly enhances its value. Fig. 129 shows pot cultures of wheat in pure sand enriched with 20 grams of dried activated sludge (Pots 3 and 5) and with equivalent amounts of nitrogen in the form of sodium nitrate (Pot 7), ammonium sulphate (Pot 9), gluten meal (Pot 11) and dried blood (Pot 13), 35 days after planting. These general conclusions as to the value of sewage sludge have recently been confirmed by Nasmith and McKay (1918) at Toronto.

Copeland (1916) cites figures showing that sedimentation, Imhoff and septic sludge contain 1.2–3.0 per cent of nitrogen, while activated sludge contains 4 to 4.5 per cent. He estimates that the dewatering and drying of the latter will cost $8–

$12 per ton of material containing 10 per cent moisture, and that the material should be worth $9–$15 per ton.

The 1916 Report of the Sewerage Commission of Milwaukee paints an even rosier picture. The Commission concludes " that activated sludge can be successfully dewatered and reduced to a fertilizer base at an expense less than its value, and that there

Fig. 129. Pot Cultures of Wheat Fertilized with Activated Sludge (3 and 5), Sodium Nitrate (7), Ammonium Sulphate (9), Gluten Meal (11), and Dried Blood (13), courtesy of Edward Bartow.

is ample available market for all we can produce." The sludge in the Milwaukee experiments was dewatered to 75 per cent moisture with a Berrigan press, an amount equivalent to half a ton of dry sludge being produced per million gallons of sewage treated at a cost of $1.73. Drying and grinding added a cost of $1.59 more per million gallons (or per half ton of dry sludge). The final product contained about 5 per cent of NH_3 (4.1 per cent of N), giving the sludge an estimated value of $10–$17.50 per ton. The dry sludge contained 52.5 per cent of organic matter, 5.1 per cent of NH_3, 5.3 per cent of fat and 1.5 per cent of phosphoric acid of which one-third was soluble. The reason for the low content of fats in activated sludge is not clear. Fats are usually held to be highly resistant to bacterial action but in this process it appears that they must be actively decomposed.

The method of handling the activated sludge used at Milwaukee involved first, concentration of the sludge from 98–99 per cent moisture to 95–97 per cent moisture by secondary sedimentation; second, dewatering by the use of presses to about 70 per cent; and finally, drying by direct or indirect heat dryers to 10 per cent.

The activated sludge produced at Cleveland had a moisture content varying between 97.5 and 98 per cent and contained, on the dry basis, 49 per cent of organic matter, 3.2 per cent of nitrogen (of which 1.6 per cent was in available form), 6.8 per cent of fats, 1.7 per cent of phosphates (calculated as P_2O_5) and 0.3 per cent of potash. The relatively low aeration provided at this plant probably accounts for the fact that the nitrogen content is materially less than that recorded at other places.

It was found at Cleveland that this sludge could be concentrated from 98 per cent to 96 per cent by secondary sedimentation and easily dewatered on a covered sand bed. The moisture content at the end of 4 days was about 80 per cent and at the end of 9 days, 75 per cent. The sludge could generally be removed with a fork at the end of 6 days. The cost of this treatment however would be prohibitive on account of the large area required. Centrifugal dewatering in a laundry type centrifuge was only moderately successful and it was concluded that pressing was probably the most practical procedure. The drying of sludge alone in a steam dryer proved somewhat unsatisfactory on account of the balling up of the material, but when mixed with equal proportions of garbage tankage the sludge was dried readily. The gross value of the sludge was estimated at $8.20 per ton (dry basis) or $5.50 per million gallons.

Pearse and Richardson (1917), as a result of their study of stockyards sewage sludge, recommend quiescent settling which they believe will concentrate packinghouse activated sludge 50 per cent, filter pressing and heat drying. They estimate the cost of pressing at $5.72 per dry ton and the cost of drying at $2.45 per dry ton. The figures are based on the use of Berrigan presses (dewatering to 75 per cent moisture content) and Ruggles-Coles direct heat dryers (drying to 10 per cent moisture content). They estimate that the dried sludge will average 5 per cent of ammonia and will be worth $12 per ton.

Finally we may cite Metcalf and Eddy (1916) who estimate

the theoretical value of activated sludge per ton of dry material at $8.10 for nitrogen, $2.20 for phosphoric acid and $4.50 for grease, a total of $14.80. They believe, however, that the grease will hardly pay for recovery; and, estimating the cost of dewatering and drying the sludge at $9.22 per ton, the nitrogen and phosphoric acid should yield a net profit of $1.08 per ton, — a much more conservative estimate than that of Hatton.

Fuller (1918) in his Indianapolis studies has called attention to the possibility of recovering valuable fertilizing materials from the sludge deposited by humus tanks subsequent to treatment of sewage in trickling beds. He believes that this material may be as valuable as activated sludge from a manurial standpoint. In the Milwaukee studies humus tank sludge had a lower water content than activated sludge (96.1 per cent) and an even higher nitrogen content (6.3 per cent on the dry basis). Worcester analyses have however failed to show any such high nitrogen values, the humus tank sludge like that from other sedimentation processes containing about 3 per cent of nitrogen on the dry basis.

The Recovery of Grease from Sludge After the Treatment of Sewage by the Miles Acid Process. In the attempt to use sewage sludge as a fertilizer the fats, which make up about 10 per cent of its solid constituents, have presented a serious difficulty, since they are not generally utilizable by plants or bacteria and tend to clog the soil and lower its agricultural value. The fats on the other hand have a commercial value of their own and it has been assumed in certain instances that they could be extracted and sold without cost or perhaps at a slight net profit, leaving the fertilizing material in a more utilizable state.

At a few cities in England and Germany the attempt has been made to extract grease from sewage sludge on a commercial scale, but generally without marked success.

At Cassel, Germany, a contract was made by the city with a firm which agreed to dispose of sedimentation sludge with a water content of 90 per cent. The process involved treatment with a rolling screen, heating to 212° F. with sulphuric acid, filter pressing while hot, drying, extraction with benzine, steam distillation of the extract, drying the grease and finally distilling it. Sixty-five cubic yards of wet sludge yielded 6.5 cubic yards of cake assumed to contain 1650 pounds of crude grease

and 10,750 pounds of sludge. The latter was estimated to have a fertilizer value of 32.5 cents per 100 pounds or $34.88 The crude grease was assumed to yield 990 pounds of refined grease worth $48.20 (at $4.87 per 100 pounds) and 495 pounds of tarry residue worth $2.05 (at 41.6 cents per 100 pounds). In spite of the total estimated value of over $1.30 per cubic yard of wet sludge the expenses of operation were greater than the receipts and the plant was abandoned (Hopfner and Paulmann 1902). Great difficulty was experienced in marketing the grease on account of its unpleasant odor.

The most famous plant for the recovery of grease from sewage is that at Bradford, England. According to Allen (1917) this is the largest sludge pressing plant ever installed comprising 128 presses which deliver 3700 pounds of cake per run. The sewage of this city is an unusually favorable one for such a process, since it contains 440 parts per million of grease, largely derived from wool-washing establishments. It is first screened, then precipitated with 2.8 tons of sulphuric acid per million gallons of sewage which process yields 40 tons of sludge (80 per cent moisture) per million gallons of sewage. This sludge is screened, treated with more acid, heated to about 100° and pressed hot, to extract the grease. Part of the pressed sludge is burned, part is sold direct to farmers and part dried further and sold for fertilizer stock. The process appears in this case to have been successful, probably as a result of the high grease content of the sewage treated. Since the war the profits have been enormously increased.

The subject of acid treatment of sewage for the recovery of grease has recently been brought to the front in the United States by the studies made in Boston of the process introduced by George W. Miles described in Chapter V. Interest in this process has been specially stimulated on account of the high value of grease and glycerin resulting from the war. The acid treatment by converting soaps into fatty acids greatly increases the amount of fat present and these constituents, which tend to hamper the attempt to recover nitrogenous material from activated sludge, become the primary object of the Miles process.

In the Boston experiments reported by Weston (1916) 1800–1900 pounds of dry sludge per million gallons of sewage were obtained by this process and of this material 22–23 per cent

was grease, the yield of grease per million gallons of sewage treated varying from 430 to 436 pounds. The sludge had a low moisture content (85.8 per cent). E. S. Dorr, who conducted the first Boston experiments, estimated the value of the grease present at $15.08 and the value of the fertilizer base at $9.25 per million gallons of sewage treated. With an estimated cost of $18 per million gallons for treatment this would yield a net profit of $6 per million gallons Prof. R. S. Weston believes that this estimated margin of profit is too high, but " is unable, by any reasonable comparison with analogous cost data from other sources, including his experience, to wipe it out."

This method of sewage treatment has been most extensively investigated at the New Haven Sewage Experiment Station during the years 1917 and 1918. The general results of these studies have been reviewed in Chapter V, and the data obtained in regard to the amount and general composition of the sludge produced are given in Table CXII below.

TABLE CXII

CHARACTER OF MILES ACID SLUDGE AT NEW HAVEN

	East St. sewer.			Boulevard sewer.	
Length of run	25 days	24 days	44 days	70 days	29 days
Total gallons sewage treated.	260,000	239,400	407,820	602,220	145,500
Gallons wet sludge per m.g. sewage	3,750	4,025	3,200	2,600	5,375
Specific gravity	1.067	1.048	1.054	1.061	.....
Per cent moisture	86.6	88	86.3	85.7	92.5
Pounds dry sludge per m.g. sewage	503	483	439	368	403
Ether extract, per cent dry sludge	23.7	24.0	29	32.6	30.9
Ether extract, pounds per m.g.	119	116	127	120	124
Volatile matter, per cent dry sludge	47.2	51.2	57.3	63.8	78.5
Nitrogen, per cent dry sludge.	1.6	1.6	2.4	2	.....

In comparing these results with those of the B ston experiments, it will be noted that the yield of sludge was less than one-fourth of that recorded at Boston, but this was to be expected since the New Haven sewage is a weak one and at stated on page 131 the amount of acid needed for precipitation was correspondingly reduced (at least with the East St. sewage). The

moisture content of the New Haven sludge was not quite so low as that obtained in Boston, but showed an even higher grease content, exceeding 30 per cent of the dry material in two of the tests.

So far the results of the New Haven experiments were very favorable to the Miles process; but when the grease which had been recovered was studied with more care in order to determine its real commercial value the aspect of the matter began to change. The difficulty lies primarily in the presence of a large proportion of unsaponifiable material (waxes, mineral oils and similar substances) in the ether extract, substances of this kind being practically worthless and their presence necessitating costly processes of purification. The grease recovered from the sludge of the third 44-day run (see Table CXII) gave the following results when analyzed by Dr. Raymond Wells:

Moisture and volatile matter	11.0
Unsaponifiable material	21.1
Free fatty acids (by weight)	40.2
Neutral grease	22.3
Insoluble and metallic soap	3.3

The grease obtained from the other three runs made with the East St. sewage as analyzed by Dr. F. W. Mohlman, chemist of the New Haven Experiment Station, contained respectively, 19.8 per cent, 20.7 per cent and 28.3 per cent of unsaponifiable material.

In view of the fact that the East St. sewer receives large contributions of mineral oil from a munition factory it was thought that the large amount of unsaponifiable material might be due to this cause and a small temporary plant was therefore installed on the Boulevard sewer which carries a fairly normal domestic sewage. The result here was distinctly better, the proportion of unsaponifiable material being only 14.5 per cent but this value is still so high as very seriously to impair the value of the grease.

It seems probable that a fairly high content of unsaponifiable material is a normal characteristic of grease obtained from sewage sludge. Thorpe in his Dictionary of Chemistry says " sewage fats are characterized by large proportions of free fatty acids. The amount of unsaponifiable matter is also considerable. The nature of this has not yet been investigated. Probably it con-

sists to a large extent of coprosterol which forms an important constituent of excrementitious matter." Lewkowitsch in The Technology and Analysis of Oils, Fats and Waxes notes the presence of 11.6 per cent of unsaponifiable material in the grease obtained at Cassel. The source of this material is apparently the feces themselves, for a review of the literature on the fat content of feces made by Mohlman in connection with the New Haven studies shows that of the ether extract of dried feces (which amounts to 25–35 per cent) 12–16 per cent is unsaponifiable matter, about half of the latter perhaps being cholesterin.

The usual limit for unsaponifiable matter in grease to be used for soap-making is about 5 per cent, and unless grease containing 10–20 per cent of material of this kind could be economically distilled it could be used only as wool grease, which is worth about half as much as garbage grease or 5–6 cents a pound according to the high prices of 1918.

In view of these facts it is still questionable whether the acid treatment of sewage for the recovery of grease would prove an economical process except where a low alkalinity of the sewage or a high content of true grease (such as obtains at Bradford) introduces specially favorable conditions. At New Haven, Conn., this process has been recommended, as the result of extensive experiments discussed above, the cost of acid treatment in this case being estimated as less than that of Imhoff treatment and chlorination for a sewage containing acid and antiseptic industrial wastes.

Destructive Distillation of Sludge Cake. Comparatively little attention has been paid as yet to this subject. Experiments, however, have been made in America and England regarding the products formed when sludge is heated in retorts, and they have shown that a large part of the nitrogen of the sludge is recovered in the form of ammonia, and that, as with the destructive distillation of organic matter in general, the other products are a more or less luminous gas, tar, oil, and a coke-like residue.

In England, George Watson (R. S. C., 1908) reports the results obtained from an experimental plant. The average amount of ammonium sulphate obtained in six different experiments from one long ton of absolutely dry sludge was 105.9 lbs., maximum 124 lbs., minimum 81.7 lbs.; the amount of residue left in the retorts averaged 47 per cent, maximum 50, minimum

42 per cent. The amount of oil was very variable, depending on the heating of the retort, the maximum amount obtained being 30 gallons. The gas was not measured, but was used for heating the retorts.

In America, the Massachusetts State Board of Health (1910) have made experiments in destructive distillation, using sludge from chemical preciptation tanks, plain sedimentation tanks, septic tanks, and settling tanks receiving the effluent of trickling filters. The distillation process was carried out on two or more samples from each source, four hundred grams of dried sludge being used in each case.

The results obtained are tabulated as follows:

TABLE CXIII

ANALYSES OF SLUDGES USED FOR DESTRUCTIVE DISTILLATION, PER CENT OF COKE FORMED AND AMOUNT OF NITROGEN IN COKE

(Mass., 1910.)

	Composition of sample before distillation (per cent).			Per cent of coke produced.	Per cent of nitrogen.*		Per cent available P_2O_5 in coke.
	Total nitrogen.	Loss on ignition.	Fats.		Found in coke.	As NH_3 in washer.	
Lawrence sludge †.....	3.36	36.8	12.8	63.5	0.11	0.586	1.33
Andover sludge †.....	2.14	46.6	27.5	59.5	0.67	0.226	1.33
Clinton sludge.....	2.36	74.4	7.7	44.5	0.72	0.404	1.44
Brockton sludge †.....	1.76	46	6.2	60.5	0.94	0.137	1.17
Worcester sludge ‡.....	1.19	44.5	3.2	54	0.09	0.544	1.67
Septic tank sludge.....	2.46	47.9	8.3	68.5	0.27	0.497	1.15
Trickling filter sludge.....	2.10	48.3	4.9	62	0.56	0.809	1.31
Sludge from evaporation of sulphite pulp liquor.....	....	87.3	....	32	....	....	....
Peat.....	2.54	92	....	49	0.70	0.700	0.31
Sawdust.....	....	....	....	25	....	....	....
Wood pulp.....	....	....	....	25	....	....	....
Soft coal §.....	....	96–8	....	77.3	....	0.222	....

* Per cent by weight of total sludge taken.

† Settled sewage sludge.

‡ Chemically precipitated sludge.

§ Average of four kinds of steam and gas coal.

RELATIVE VOLUME AND COMPOSITION OF GASES PRODUCED BY DESTRUCTIVE DISTILLATION OF SEWAGE SLUDGE

	Cubic feet of gas per ton of sample.	Per cent.						
		CO_2	Illuminants.	O	CO	H	CH_4	N
Lawrence sludge *	4,900	4.4	2.2	0.3	30.7	34.9	18.6	9.1
Andover sludge *	6,400	7.4	15.1	0.6	14.3	22.9	34.3	5.4
Clinton sludge *	9,100	8.3	6.7	...	20.4	33.2	24.5	7
Brockton sludge *	6,000	16.5	21.4	0.2	10.3	22.6	29.1	0.2
Worcester sludge †	8,100	14.2	4.9	0.3	29.8	32.6	16.2	2.2
Septic tank sludge	4,900	7.5	1	0.1	24.3	44	13	10.2
Trickling filter sludge	6,000	20.2	17.4	0.3	6.6	32.7	22.8	....
Sludge from evaporation of sulphite pulp liquor	11,000	21.6	2.1	....	20	42	12	0.3
Peat	8,400	39	4.7	0.2	11	28	17.1	....
Sawdust	12,700	18.4	4.8	0.3	28.2	26.9	19.1	2.3
Wood pulp	12,000	23.5	1.4	....	16.4	44.5	13.3	0.9
Soap grease	5,400	6.8	44.5	....	6.2	15.8	26.7	....
Soft coal ‡	10,200	1.6	2	0.1	5.2	62.3	25.7	3.2
Lawrence illuminating gas §	.....	3.4	9.1	....	21.5	42.5	19.7	3.8

* Settled sewage sludge.
† Chemically precipitated sludge.
‡ Average of four kinds of steam and coal.
§ From gas company pipes at experiment station

Cost and Availability of Various Methods of Sludge Disposal. Very few data are obtainable as to the cost of sludge disposal in America, by the various methods described. A rough comparison as to the relative cost of sludge disposal in England can, however, be obtained from Table CXIV (R. S. C., 1908).

Though, as has been stated, the method which should be chosen for the disposal of sludge depends on local conditions, as, for instance, the size of the plant and its location, yet the following general statements can be made:

For small plants not situated in the near vicinity of dwellings, land drying or lagooning is, as a rule, advisable, the dried sludge being given to farmers or disposed of by digging it into the soil. Drying under glass will frequently be preferable to drying in the open. For medium-sized plants, where ample land is available trenching may prove satisfactory since it avoids the creation of a nuisance and the rehandling of the sludge. For large plants, if situated near the coast, sea disposal will usually be found the best and least costly method. For large inland plants mechanical drying must be resorted to, with subsequent disposal by gift to farmers, by digging into the soil, or by filling up of low land in very isolated localities, and where these methods are not avail-

TABLE CXIV

COMPARATIVE COST OF THE VARIOUS METHODS OF SLUDGE DISPOSAL, AS INSTANCED BY THE EXAMPLES GIVEN

(R. S. C., 1908.)

Method of sludge disposal.	Number of examples.	Maximum and minimum cost of the process, in cents, per ton of sludge containing 90 per cent of water, including interest and sinking fund and all other charges.		Average cost of process, in cents per ton of sludge containing 90 per cent of water, including interest and sinking fund and all other charges.
			Cts.	Cts.
Covering land with sludge	3	Minimum	2.64	4
		Maximum	5.6	
Sea disposal.....	6	Minimum	8.2	10
		Maximum	13.8	
Trenching in soil.....	3	Minimum	8	10
		Maximum	14	
* Pressing, Group I....	10	Minimum	9.6	12
		Maximum	14.6	
† Pressing, Group II...	11	Minimum	15.4	23
		Maximum	25.2‡	
Pressing and burning ..	1	excluding interest and sinking fund	26.6	36§

* In this group are included towns having a population of about 30,000 and upwards, where the preliminary treatment consists either in chemical precipitation followed by sedimentation or in simple sedimentation, and where the sewage does not contain manufacturing waste of a kind likely to necessitate the addition of an unusual quantity of lime to the sludge before pressing.

† In this group are included towns having a population under 30,000 and also places where 5 to 20 per cent of lime (calculated on the pressed cake) has to be added to the sludge before pressing, either because the sewage contains much grease, or because septic tank sludge has to be dealt with.

‡ Owing to the great variations in the actual figures for interest and sinking fund, mainly because of the different dates at which the sewage works were constructed, we have been obliged, in dealing with "Pressing," to take an approximate average figure for this item, *viz.*, 18 cents per ton of pressed cake containing 55 per cent of water, or 4 cents per ton of sludge containing 90 per cent of water.

§ This figure is an estimate.

able, by burning it, mixed with house refuse or a small amount of fuel, in a destructor.

Where the sludge contains certain kinds of trade waste, as, for instance, wool grease at Bradford, England, special methods of treatment for the recovery of grease may be advisable; while with activated, and perhaps also humus tank, sludge it will be cheap and perhaps even actually profitable to heat-dry the dewatered sludge and convert it into fertilizer stock.

CHAPTER XIV

DISINFECTION OF SEWAGE AND SEWAGE EFFLUENTS

Conditions under which Bacterial Purity of Sewage Effluents may be Necessary. The primary object of sewage purification is the oxidation of organic matter — its conversion into a stable form, so that it will not putrefy and create a nuisance. This end can often be attained by methods which do not effect a very notable reduction in the number of bacteria present. In many cases such methods are well suited to local conditions. Where, for example, the effluent from a purification plant is discharged into a small stream which is not used as a source of water supply, organic stability and removal of suspended matter is all that can properly be demanded. In other instances, however, bacterial purification may be required. Where an effluent is discharged into the ocean in a strong current of water, suspended solids and organic matter may prove inoffensive; but if there be important shellfish beds in the neighborhood, bacterial purification may be an imperative necessity.

The relative importance of freedom from solids, organic stability and bacterial purity must be determined in each particular case by a study of local factors. The principal conditions which indicate the need for an effluent, low in bacteria, may be briefly summarized as follows:

a. When a stream is used without purification as a source of water supply the removal of bacteria at some point is obviously essential, and the same is true, in the case of sewage discharged in any large amount into lakes. It is generally far more economical, however, and it is always far more efficient to purify the water taken out of a stream or lake for potable purposes than to purify the sewage which discharges into it. Even if all direct contributions from public sewers are eliminated surface waters usually require treatment before they can be safely used for drinking, on account of the danger of pollution washed in from various points on their banks. Sewage treatment can rarely, if ever, be wisely used as a *substitute* for water purification.

b. Even when the water taken from a stream is purified before it is used, it may be desirable to remove bacteria at a sewage outfall above, if the water intake is in the near neighborhood of the sewage outfall and if the amount of pollution is considerable. The character of a raw water determines the rate of filtration and the cost of the process. It is not fair to place an abnormally heavy burden upon one community as a result of the negligence of another.

The International Boundary Commission, as a result of its study of pollution in the Great Lakes, suggested that the raw water to be treated by a purification plant should contain not over 500 *B. coli* per 100 c.c. as an annual average, and should not show *B. coli* in more than half of a series of 0.1 c.c. samples if satisfactory results are to be secured. According to this standard it would naturally be required that sewage treatment should be carried far enough so that after dilution has taken place the effluent discharged shall not produce, at any neighboring water works intake, a raw water of greater bacterial impurity than that specified above. The requirement of the Commission is perhaps an unduly strict one since many filtration plants (such as that of Cincinnati) are treating waters much more polluted than this with success.

c. It may at times be proper to purify sewage bacterially before it is discharged into a body of water used extensively for bathing.

d. It is probably best under all conditions to provide sterilizing treatment for specially infected sewage (like that from a contagious disease hospital) before it is discharged into any stream.

e. The most important case, in practical magnitude, is the case in which shellfish layings are threatened by the discharge of sewage into tidal waters. In this case, — unless the processes of purifying shellfish by storage in clean disinfected water suggested by the U. S. Public Health Service, are introduced on a practical scale, — there may be no alternative except the abandonment of the shellfish industry or the bacterial purification of the sewage which menaces it. All along the Atlantic seaboard this is a problem of pressing moment.

The Bacterial Efficiency of Sewage Filtration. There is a wide difference in bacterial efficiency between the older processes of

sewage disposal, irrigation and intermittent filtration, on the one hand, and the newer processes of the contact bed and the trickling filter, on the other. Figures have been quoted in Chapter VIII (page 217) which show that, while most of the English sewage farms yield effluents containing bacteria in the hundred-thousands, the Nottingham effluent shows frequently less than 1000 bacteria per c.c. In Chapter IX it has been shown that good intermittent sand filters yield still better results. At Ames, Iowa (page 266) and at Brockton, Mass. (page 267), the bacterial reduction is well over 99 per cent and the actual number of bacteria in the effluents is generally under 10,000 per c.c. Such results as these indicate that the intermittent sand filter, when properly constructed and properly operated, will secure a reasonable bacterial purification as well as an organically stable effluent.

Activated sludge treatment can be so applied as to secure a material bacterial reduction. The average bacterial content of the sewage treated at Milwaukee for the eight months, January–August, 1916, was 1,028,000 per c.c., while the average content of the effluent was 90,000 per c.c., a purification of 91 per cent.

With the various types of coarse-grain filters, conditions are very different from those which obtain in intermittent filtration. Such processes produce a certain reduction in bacterial numbers, it is true; but the decrease is not sufficient to be of very great sanitary moment. With a material containing so many bacteria as sewage, a reduction of 99 per cent may be satisfactory, but a reduction of 90 per cent, leaving several hundred thousand bacteria in the effluent, is usually not. That no better result than this can be expected from the contact bed or trickling filter may be made clear by a few figures quoted from the cases collected by Prescott and Winslow (1908):

"In the Columbus experiments, Johnson (1905) found from one to two million bacteria in the effluents of contact beds and from 750,000 to 1,900,000 in the effluents from trickling filters. The average percentage reduction effected by seven contact beds and six trickling filters is shown below:

TABLE CXV

REDUCTION OF BACTERIA AT COLUMBUS, OHIO

(Johnson, 1905.)

Contact beds.	Per cent reduction.	Trickling filters.	Per cent reduction.
Primary A	60	A	74
Primary B	43	B	70
Primary C	33	C	70
Primary D	33	D	69
Primary E	0	E	46
Secondary A	38	F	21
Secondary B	39	.....	.....

" Thumm and Pritzkow (1903), at the Berlin Experiment Station, obtained the results tabulated below:

TABLE CXVI

BACTERIA IN SEWAGE, CONTACT EFFLUENT AND SAND EFFLUENT AT BERLIN

	Bacteria per c.c.
Crude sewage	16,900,000
Primary contact effluent (coarse coke)	12,400,000
Secondary contact effluent (fine coke)	5,600,000
Tertiary sand effluent	1,100,000
Primary contact effluent (fine coke)	7,400,000
Secondary sand effluent	1,800,000

" At the experiment station of La Madeleine, in Lille, Calmette (1907) reports 5,000,000 bacteria per c.c. in the crude sewage, 2,900,000 in the second-contact effluent and 800,000 in the effluent from the trickling bed. Of 20,000 *B. coli* per c.c. applied to the filters, the contact system delivered 4000 and the trickling bed 2000 per c.c.

" The average results of examinations made three times a week at the Sewage Experiment Station of the Massachusetts Institute of Technology, during two different periods, were as follows:

TABLE CXVII

BACTERIA IN SEWAGE, SEPTIC EFFLUENT AND TRICKLING EFFLUENTS AT BOSTON

(Winslow and Phelps, 1907.)

	Bacteria per c.c.				*B. coli* positive tests in 0.000001 c.c.*
	July–Sept., 1906.		Oct., 1906–April, 1907.		July–Sept., 1906.
	No.	Per cent reduction.	No.	Per cent reduction.	Per cent positive.
Sewage	1,300,000	.....	1,200,000	.....	65
Septic effluent	1,650,000	Inc.	750,000	38	66
Effluent from trickling bed	750,000	42	200,000	83	35
Effluent from septic tank and trickling bed	750,000	42	180,000	85	35

* Jackson bile test.

" The contact beds, as operated on a practical scale in England, show considerably higher numbers. At London the Barking and Crossness beds yielded effluents containing one to five million bacteria per c.c., of which 100,000 to 600,000 were *B. coli*.

" There are few plants of the newer types now in operation in the United States, and fewer still are controlled by bacteriological examinations.* At Plainfield, N. J., however, the combination of septic tank and double-contact beds produces a bacterial purification of 80 to 90 per cent as measured by total numbers. The following table shows the results of four examinations made in 1906:

TABLE CXVIII

BACTERIA IN SEWAGE, SEPTIC EFFLUENT AND CONTACT EFFLUENT AT PLAINFIELD, N. J.

(N. J., 1907.)

Date.	Sewage.	Bacteria per c.c.		*B. coli* in —		
		Septic effluent.	Secondary contact effluent.	Sewage.	Septic effluent.	Secondary contact effluent.
				c.c.	c.c.	c.c.
July 9	2,295,200	659,200	591,300	0.00001	0.00001	0.0001
July 9	2,043,300	555,000	172,600	0.000001	0.0001	0.0001
August 9	1,371,700	989,700	186,700	0.00001	0.0001	0.0001
August 9	1,655,000	.....	338,000	0.000001	.....	0.00001

* Figures for Worcester and Reading are cited on pages 368 and 370.

" It is obvious that effluents of this character cannot be considered satisfactory from the standpoint of bacterial purification. As Houston concluded, after a careful review of the subject, ' The different kinds of bacteria and their relative abundance appear to be very much the same in the effluents as in the crude sewage. Thus, as regards undesirable bacteria, the effluents frequently contain nearly as many *B. coli*, proteus-like germs, spores of *B. enteritidis sporogenes* and streptococci, as crude sewage. In no case, seemingly, has the reduction of these objectionable bacteria been so marked as to be very material from the point of view of the epidemiologist ' (Houston, 1902).

" Experimental studies with specific bacteria have confirmed these conclusions. Houston (1904*b*) found that *B. pyocyaneus* appeared in the effluent of a trickling bed ten minutes after application to the top and continued to be discharged for ten days. In septic tanks and contact beds, the same germ persisted for ten days. Rideal (1906) quotes experiments by Pickard at Exeter, which show that typhoid bacilli may persist for two weeks in a septic tank and that contact bed treatment only effects a 90 per cent removal of these organisms."

Clearly, where bacterial purity is required, the contact bed and the trickling filter must be supplemented by some special process of bacterial removal.

Possible Methods of Disinfecting Sewage. A wide variety of processes have been suggested for disinfecting sewage and sewage effluents, particularly in England. Most of them are discussed with some fullness by Rideal (1905), and with calculations as to American costs by Phelps and Carpenter (1906). They may be grouped for convenience under five heads: (1) heat; (2) caustic lime; (3) acids; (4) metallic salts; (5) oxidizing agents.

1. *Heat.* The use of heat has been suggested by Klein (R. S. C., 1902). The sewage was to be treated by a patented apparatus with the recovery of ammonia. Phelps and Carpenter calculated that to raise sewage to the boiling point would require 40 tons of coal per million gallons, at a cost of $160. At present this process is certainly not practical, although recent advances in the sterilization of drinking water by heat have shown that surprising economies may be effected by properly designed apparatus.

2. *Caustic Lime.* Caustic lime has an important application in sewage purification, for the removal of suspended solids, as pointed out in Chapter V. It is one of the best precipitants in use, and in the course of precipitation a large number of bac-

teria are mechanically removed. Lime has also a certain direct disinfectant action. Rideal found however that 1000 parts per million failed to produce satisfactory bacterial purification.

3. *Acids.* Acid substances, as a class, exert a much stronger disinfectant action than alkalis. Some have a specific poisonous effect of their own, like carbolic and benzoic acids; and the mineral acids uniformly exercise disinfectant properties by means of their dissociated hydrogen ions. Winslow and Lochridge (1906) have shown that 0.005 normal solutions of hydrochloric acid or sulphuric acid are fatal to typhoid bacilli in tap water, in ten minutes.

The possibility of acid disinfection has been revived in a practical form by the development of the Miles Acid process for the precipitation and disinfection of sewage and the recovery of grease, which has been discussed in Chapter V. In the Boston experiments reported by Weston (1916) an average count of 4,340,000 bacteria per c.c. (20° count) was reduced to 827 per c.c. by treatment with 2300 pounds of sulphur dioxide per million gallons of sewage. The results in Table CXIX were obtained at New Haven in a longer series of experiments. The sewage of the East St. sewer is an abnormal one, showing a very low bacterial count as a result of the presence of copper salts contributed by industrial wastes. The figures cited in the table for this sewer are for Sunday and 8 A.M. samples only, so that the effect of this abnormality is minimized so far as possible.

TABLE CXIX

BACTERIAL RESULTS OF MILES ACID TREATMENT

(New Haven, Conn.)

Sewer.	Raw sewage.		Treated effluent.			
	Bacteria per c.c., 20° 3 day count.	*B. coli* per c.c.	Bacteria per c.c., 20° 3 day count.	*B. coli* per c.c.	Per cent of all samples with less than	
					10,000 bacteria per c.c.	1000 *B. coli* per c.c.
East St.....	483,000	24,000	20,800	560	91	84
Boulevard.....	1,879,000	221,000	7,325	2,305	91	41

If the recovery of grease should prove in the future to be a practicable procedure (which at present as pointed out in the last chapter is not certain), the acid disinfection of sewage may deserve very serious consideration.

4. *Salts of Heavy Metals.* The salts of the heavy metals are stronger disinfectants than the acids. In their case, however, the presence of organic matter exerts an inhibition and makes the use of large quantities necessary. Metallic salts are generally costly in themselves, so that their use is rarely feasible with a substance of high organic content like sewage. Copper is the particular metal which has been suggested in this connection, from the success with which copper sulphate has been used in freeing water supplies from Algæ; but copper is relatively stronger as an algicide than as a bactericide. Johnson and Copeland (1905), in experiments at Columbus, obtained the results tabulated below; and according to these estimates a good bacterial reduction could be attained at a cost of $5 to $10 per million gallons. Other methods discussed later on, are as efficient at a less cost.

TABLE CXX

DISINFECTION OF SEWAGE EFFLUENTS BY COPPER SULPHATE

(Johnson and Copeland, 1905.)

Series.	Copper sulphate, parts per million.	Per cent reduction.	
		Three hours.	Twenty-four hours.
I	5	90	99.90
	10	98	99.95
	20	98.5	99.96
II	10	40	99.70
	20	60	99.90
	40	88	99.95

5. *Oxidizing Agents.* The fifth class of disinfectants includes the oxidizing agents, of which three have been practically used in water and sewage work — ozone, permanganates or manganates, and compounds yielding chlorine.

Ozone plants have been installed with more or less success for the sterilization of water at various small towns in Europe. There is no doubt as to the efficiency of the process with water, but the cost has generally been high and the apparatus subject

to serious and expensive breakdowns. Methods of preparing ozone are being constantly improved and cheapened; but it can scarcely be considered a practical working process for sewage treatment. The organic matter present would use up large amounts of the gas, and even if the supply were ample its slight solubility makes it doubtful whether even small solid ma ses of suspended matter would be adequately penetrated.

Permanganates and manganates yield oxygen both in acid and alkaline solutions, and the oxygen being in the nascent or potential state, these substances act as germicides. This germicidal action, however, is much less marked than that of either ozone or chlorine and when used in sewage treatment it has been rather for the destruction of organic matter than for sterilization. At one time the sewage of London was treated with chlorine to destroy the obnoxious odors, but Dibdin, as chemist of the London County Council, substituted potassium permanganate, for the express purpose of supplying oxygen, without producing sterilization, as he found that the sewage which had been partially sterilized with chlorine underwent a secondary putrefaction of a particularly offensive nature, due probably to partial destruction of nitrifying organisms.

Where disinfection is specifically aimed at, chlorine seems by far the most effective of all the oxidizing agents. It is the only one which at the present time is being practically used for the sterilization of sewage effluents.

The Use of Chlorine for the Disinfection of Sewage. Chloride of lime and bleaching powder have long been used as deodorants and disinfectants for the direct treatment of excreta; but the application of similar disinfectants to sewage is of recent date.

The active constituent in a solution of bleaching powder or calcium hypochlorite is CaO_2Cl_2, which is rapidly decomposed so as to yield hypochlorous acid, the latter being immediately broken up into hydrochloric acid and oxygen. It is this nascent oxygen which is the actual disinfecting agent. That portion of the total chlorine present in the bleaching powder, which is present in the form of CaO_2Cl_2 and is therefore capable of liberating nascent oxygen, is called "available chlorine"; and it usually amounts to about one-third of the total weight of the bleach.

A number of years ago, in certain methods of sewage treatment, such as the Webster process, the Hermite process, and the Woolf process, hypochlorites were a more or less essential feature. A plant of this type was installed at Brewster, N. Y., in 1875. None of these processes were commercially successful, however, and until about 1910 it was generally considered by sanitary engineers that though chlorine disinfection might be useful for special emergencies, it was far too costly for a routine method of treatment.

The view that chlorine disinfection was only suitable for special emergencies, and perhaps for the treatment of hospital sewages, was largely due to the work of German investigators (Schumacher, 1905; Kranepuhl, 1907; Kurpjuweit, 1907) who reported that very large amounts of chlorine were necessary for treatment. The reason for this conclusion was that these workers insisted on an extreme degree of purification, the criterion of success being the absence of *B. coli* in a large proportion of cases from one-liter samples. It was not realized that while absolute sterilization required large amounts of chlorine a reasonable degree of purification could be attained with very much smaller quantities. The recognition of these facts brought about a revolution in the art of sewage disinfection.

The credit for the first step in this demonstration belongs to Rideal. He showed in his experiments at Guildford (Rideal, 1905) " that 30 parts of available chlorine per million would reduce the number of bacteria in crude sewage from several millions to 50,000, while 50 parts would reduce their number to 20 per c.c. Colon bacilli were reduced from one million per c.c. to less than one per c.c. by 30 parts of chlorine. In septic effluent 25 to 44 parts of chlorine per million reduced *B. coli* from two and a half to four and a half million per c.c. to less than one per c.c. With contact effluents smaller amounts of chlorine proved efficient. The primary effluent required 20 parts per million, the secondary effluent 10.6 parts per million and the tertiary effluent 2.5 parts per million to reduce the number of *B. coli* so that this organism could not be isolated in 5 c.c."

Almost immediately these results were confirmed and extended by a series of investigations carried out in this country at the Sewage Experiment Station of the Massachusetts Institute of Technology in Boston. Phelps and Carpenter (1906) found in

preliminary laboratory experiments that with fairly well-purified effluents derived from more dilute American sewages even better results could be attained than those recorded in Rideal's experiments. A series of tests carried out during the summer of 1906 with the effluents from the trickling filters at the experiment station gave the results tabulated below.

As a result of these tests and later ones, the conclusion was reached that trickling filter effluent could be disinfected by the addition of hypochlorite of lime in such an amount as to yield five parts per million of available chlorine at the cost of about $1.50 per million gallons of sewage treated (Winslow and Phelps, 1907). Later experiments at the Boston station showed a con-

TABLE CXXI

BACTERIA IN TRICKLING FILTER EFFLUENT BEFORE AND AFTER TREATMENT WITH HYPOCHLORITE OF LIME

Five parts per million available chlorine. (Phelps and Carpenter, 1906.)

Date.	Bacteria per c.c.		*B. coli*, Jackson bile test.	
	Before.	After.	Before, .000001 c.c.	After, 10 c.c.
1906				
August 11	270,000	69	+ 0	+ 0
13	630,000	41	0 0	+ 0
14	135,000	406	+ +	+ 0
15	230,000	21	0 0	0 0
16	250,000	37	+ 0	0 0
18	110,000	40	0 0	+ 0
20	90,000	54	+ 0	0 0
21	220,000	22	0 0	0 0
23	.....	.....	+ 0	0 0
Average	240,000	86	33%	22%
Average removal	99.96%		999.93%	

tinued high efficiency and brought out a number of other interesting points. Table CXXII, for example, shows the effect of varying quantities of chlorine and the effect of temperature upon the amounts required. Like any other chemical reaction, the poisonous action varies with the temperature in its velocity.

TABLE CXXII

DISINFECTION OF TRICKLING FILTER EFFLUENT AT BOSTON. SUMMARY OF RESULTS AVERAGED BY PERIODS, TO SHOW THE EFFECT OF CHANGES IN TEMPERATURE AND IN THE AMOUNT OF AVAILABLE CHLORINE

(Phelps, 1909.)

Period.	Temp. Degrees F.	Available chlorine, parts per million.	Bacterial removal, per cent.				
			Bacteria at 20°.		Bacteria at 37°.		*B. coli.*
			Total.	Liquefiers.	Total.	Acid-formers.	
Nov. 12 to June 27.....	45	3.4	96.8	98.1	97.4	97.3	99.19
Nov. 12 to Dec. 12.....	42	6.3	99.6	99.7	99.8	99.9	99.99
Jan. 27 to Mar. 28.....	36	3.2	95.8	97.7	96.6	96.4	98.50
April 27 to June 27.....	60	2.9	97.1	98	97.6	97.9	99.07

Meanwhile, another important series of investigations was carried out in the state of Ohio with generally similar results. K. F. Kellerman, representing the United States Department of Agriculture, and R. W. Pratt and A. E. Kimberly, representing the Ohio State Board of Health, studied the action of chlorine and of copper on effluents of various sorts at a number of different plants in the state. They came to the conclusion that " both calcium hypochlorite and copper sulphate have high germicidal values when acting upon partially purified sewage. Calcium hypochlorite is much more rapid in its action, is more nearly able to bring about complete disinfection at a lower cost, and is less influenced by temperature and by the presence of carbonates. It is, however, liable to deterioration upon standing and is more disagreeable and less convenient to handle than copper sulphate " (Kellerman, Pratt and Kimberly, 1907). The more important results obtained with chlorine are averaged in Table CXXIII. The cost for treating sand effluent at Lancaster was estimated at $5.78 per million gallons. At Marion the estimates ranged from $8.83 for septic effluent to $2.73 for contact effluent and $2.43 for sand filter effluent. The high cost estimates are due to the proportionately greater expense of labor at small plants.

TABLE CXXIII

DISINFECTION OF EFFLUENTS WITH HYPOCHLORITE OF LIME AT LANCASTER AND AT MARION, OHIO

(Kellerman, Pratt and Kimberly, 1907.)

Series.	Available chlorine, parts per million.	Bacteria at 20°.		Bacteria at 37°.		Acid formers at 37°.	
		Initial.	Final.	Initial.	Final.	Initial.	Final.
A	4	130,000	140	14,000	49	840	0
B	2.8	60,000	1,600	12,000	120	3,000	0
C	4.1	225,000	1,600	120,000	390	16,000	1
D	6	2,000,000	700,000	900,000	230,000	70,000	24,000

Series A. Effluent of sand filter at Lancaster.
" B. Effluent of sand filter at Marion.
" C. Effluent of contact filter at Marion.
" D. Effluent of septic tank at Marion.

Later American Investigations on Chlorine Treatment. Along the coast of New Jersey are a number of small sewage-disposal plants, and more are being constructed every year. Shellfish layings are numerous along this coast, and the elimination of disease bacteria is in many cases the most important part of the process of purification. Suitable areas for sand filtration are rarely available, and septic tanks and contact beds are commonly in use. As soon as it appeared that bacterial purification could be attained at a reasonable cost by chemical treatment, Professor Phelps was called in consultation by the New Jersey State Sewerage Commission; and in the fall of 1906 experiments on a practical scale were begun at the town of Red Bank.

Red Bank has a population of 6500 or more, and the average dry-weather flow of sewage is about 265,000 gallons per day. The sewage passes through grit chambers and a circular septic tank, holding about eight hours' flow. After treatment in the tank the effluent was originally passed through strainers. The use of the water below for bathing and the proximity of shellfish beds made it imperative to secure better bacterial purification. Professor Phelps, therefore, advised that a series of experiments should be made to determine the practicability of treating the effluent with hypochlorite of lime. These were carried out during 1906 and 1907 by F. E. Daniels, acting under the direction of Professor Phelps. The general results for 1907 are indicated in Table CXXIV.

TABLE CXXIV

DISINFECTION OF SEPTIC SEWAGE WITH HYPOCHLORITE OF LIME AT RED BANK, NEW JERSEY

Bacteria per c.c. Weekly averages. (Phelps, 1909.)

Week ending.	Available chlorine, parts per million.	Total bacteria at 20°.			*B. coli* (Bile).		
		Initial.	0.75 hour.	1.5 hours.	Initial.	0.75 hour.	1.5 hours.
July 20.....	9.9	800,000	410	460	46,000	4	4
" 27.....	10.6	650,000	800	420	80,000	13	11
August 3.....	11.5	1,800,000	550	130	40,000	21	5
" 10.....	11.4	850,000	240	140	55,000	14	2
" 17.....	13	760,000	2,100	1,500	70,000	30	28
" 24.....	7.3	700,000	45,000	55,000	70 000	700	600
" 31.....	7.5	1,200,000	45,000	26,000	220,000	16,000	2,000
September 14..	11.8	750,000	13,000	8,000	300,000	150	140
" 21..	13.1	750,000	850	800	500,000	270	260
" 28..	10.5	700,000	120	88	550,000	80	28
Average *.....	11.5	900,000	2,300	1,400	210,000	75	60

* Exclusive of period, August 19–31. Temperature, 56° to 58° F. throughout.

The particular interest in these experiments lies in the fact that they were conducted with septic tank effluent. The amount of chlorine required for disinfection varies directly with the amount of oxidizable organic matter present; the Ohio experiments, cited above, show that contact effluents require more chlorine than sand effluents and that septic tank liquid requires more than contact effluent. It was therefore necessary to use large amounts of chlorine at Red Bank, about twice as much as was sufficient in the Boston experiments. The table above shows that seven parts of chlorine would not do, but that ten or eleven parts per million gave good results. The State Sewerage Commission finally recommended the application of 12–15 parts per million of available chlorine, estimated to cost $3.75 per million gallons of sewage treated (Phelps, 1908).

A little farther down the Atlantic seaboard the city of Baltimore was facing a very similar condition of affairs on a much larger scale. Baltimore had a population of a little over half a million and had recently begun the construction of a comprehensive system of sewage to remove the pollution of the Patapsco River. As soon as it was proposed to discharge all the sewage of the city at a single point the oyster industry of Chesapeake Bay was aroused, and in calling upon experts for advice as to

disposal, the Sewerage Commission specified that "the effluent proposed to be discharged into Chesapeake Bay or its tributaries in the system to be recommended by the engineers shall be of the highest degree of purity."

The Board of Advisory Engineers recommended trickling filters as a primary treatment for the sewage, and in its first report (Baltimore, 1906) suggested supplementary sand filters for accomplishing bacterial purification. The cost of the supplementary treatment alone was estimated at over a million dollars for construction and $55,000 a year for operation. When the disinfection results obtained at the Technology experiment station were published, Professor Phelps was invited to conduct experiments at Baltimore in coöperation with E. B. Whitman. The general results of these tests have been published in an exhaustive review of the whole subject, as a bulletin of the United States Geological Survey (Phelps, 1909).

The chlorine was applied, in the Baltimore experiments, to the effluent of one of the experimental trickling filters at the Walbrook Testing Station. The bleaching powder was kept down to an average of about two parts per million of available chlorine. The results were good, considering the small amount of disinfectant used; the average reduction of bacteria determined on gelatin at 20° was 95.8 per cent; of total bacteria at 37°, 94.9 per cent; of acid formers at 37°, 97 per cent; of *B. coli* as determined by the lactose bile test, 90 per cent. These results would seem to indicate the advisability of increasing the dose of chlorine to something nearer the five parts per million found necessary at Boston. On the whole, however, these experiments were considered successful. The Board of Advisory Engineers recommended the substitution of chlorine treatment for the supplementary sand treatment originally planned. The net saving to the city by the adoption of chlorine disinfection was estimated to be in the neighborhood of a million dollars.

A valuable series of studies at the Philadelphia Experiment Station (Phila., 1911) indicated (see Table CXXV) that 12 parts per million of available chlorine (300 pounds of bleaching powder per million gallons of sewage) would effect practically complete disinfection of screened or settled sewage, while half this amount with screened sewage and about one-third of this amount with

settled sewage would yield a reasonable degree of purification. These experiments are based on a two-hour detention period in a tank treating 20,000 gallons a day.

TABLE CXXV

DISINFECTION OF SCREENED AND SETTLED SEWAGE

(Philadelphia, 1911.)

Sewage effluent treated.	Available chlorine added p.p.m.	Residual chlorine in effluent p.p.m.	Bacteria per c.c. Gelatin 20°.			*B. coli* per c.c. Presumptive test.		
			Initial.	Final.	Per cent removal.	Initial.	Final.	Per cent removal.
Fine mesh screen	12.4	4.7	2,470,000	337	99.99	121,000	20	99.98
	6	0.8	2,060,000	181,000	91.21	149,000	7470	95.42
Settled sewage	11.9	3.4	2,450,000	350	99.99	143,000	10	99.99
	5.4	0.7	760,000	31,000	95.92	67,000	745	98.89
Screened and settled sewage	12	3.1	2,130,000	310	99.99	86,000	20	99.98
	4.3	1.1	660,000	22,500	96.59	317,000	1350	99.57

At Milwaukee the treatment of Imhoff effluent with liquid chlorine gave the results tabulated below so far as per cent removal of bacteria and cost of chlorine were concerned. It was assumed that in the case of sewage discharged into Lake Michigan an 85 per cent purification would probably be sufficient; and it was pointed out that while an 85 per cent purification with 5.0 parts of chlorine would cost the city $107,000 a year an increase in chlorine to 12.3 parts so as to give a 98 per cent reduction would increase the cost to $264,625.

TABLE CXXVI

EFFICIENCY OF VARIOUS CONCENTRATIONS OF LIQUID CHLORINE IN DISINFECTING IMHOFF EFFLUENT

(Milwaukee, 1915.)

Applied chlorine p.p.m.	Per cent removal of bacteria, counts at 37° C.	Cost of chlorine per million gallons.	Applied chlorine p.p.m.	Per cent removal of bacteria counts at 37° C.	Cost of chlorine per million gallons.
4.6	79.4	$2.70	10	98.7	$5.90
5	85	2.94	10.4	91	6.10
5.6	87.8	3.30	11.6	96	6.84
7.2	91.1	4.24	12.3	98	7.25
8.5	95	5	13.4	99.8	7.90
9	96.9	5.30	15.6	99.9	9.20

The prolongation of period of contact was found at Milwaukee to be important with small doses of chlorine, while where large amounts are used the chemical acts very quickly.

TABLE CXXVII

EFFICIENCY OF VARIOUS PERIODS OF CONTACT IN CHLORINE DISINFECTION

(Milwaukee, 1915.)

Per cent removal of bacteria.

Chlorine applied p.p.m.	Period of contact in minutes.		
	2.	10.	30.
5.1	71	78	82
6	78	88	94
8.5	90	94	95
9	93	99	99
14	99	99	99.9

Results tabulated below from the studies conducted at Cleveland indicate that the settled sewage of that city would require 7–12 parts of chlorine to effect an important degree of disinfection and that screened sewage (Riensch-Wurl) could not be adequately disinfected with even the largest of these amounts. The results are not strictly comparable, since the settled sewage and the screened sewage were from different parts of the city and of somewhat different composition.

TABLE CXXVIII

DISINFECTION OF SETTLED AND SCREENED SEWAGE AT CLEVELAND

Bacteria per c.c.

Applied chlorine p.p.m.	Settled sewage, east side.		Screened sewage, west side.	
	Untreated.	Treated.	Untreated.	Treated.
3	.....	.....	247,200	104,700
5	1,241,700	33,000	247,200	42,880
6	331,400	18,900	.....	.....
7	505,000	6,100	320,300	20,900
10	380,000	2,500	279,300	32,670
12	686,600	1,430	279,300	24,670

Supplementary Applications of the Disinfection Process. The primary object of chlorine treatment is of course bacterial removal and not chemical purification. In connection with other properly designed works for the purification of sewage, chlorine may, however, also be used for a certain amount of direct chemical oxidation. Experiments by Rideal in 1904–05 proved that bleaching powder when used in small quantities is sufficient to destroy foul odors, such as those of a septic effluent, and at the same time acts as a powerful oxidizer and decreases the work required of the filters. The results of a long series of experiments, from August, 1906, to August, 1907, carried out at Guildford, are summarized as follows (R. S. C., 1908):

1. Treatment with a hypochlorite solution containing available chlorine equal to 35–50 per cent of the amount of oxygen consumed by permanganate in 5 minutes in the cold is sufficient to do away with the smell of hydrogen sulphide, leaving only an inoffensive odor of spent bleach and fresh sewage.

2. The addition of this quantity in no way interferes with the efficiency of the filter, but adds to it by helping to keep down an excess of gray growths on the top.

3. A much larger quantity may be added without any danger to the filter, somewhere between 200 and 500 per cent being the limit of safety.

4. A dose of oxychloride more than sufficient to remove the hydrogen sulphide smell and kill *B. coli* in the liquid may be used without prejudice to the purifying ability of a mature percolating filter.

The author draws these further conclusions: (1) that sewage from hospitals may be freed from dangerous organisms by the use of oxychloride before passing into beds or entering ordinary sewerage systems without interfering with the usual methods of purification; (2) that when beds are clogged with growths these can be dissolved and washed through speedily by occasional doses of oxychloride. From a third series of similar experiments with each filter divided into two compartments, one containing a fine and the other a coarse filtering medium these conclusions are drawn: (1) that a bed can be successfully matured when using oxychloride in such quantities as are required for preventing offensive odors; (2) that this treatment renders possible the use of fine grade filters; (3) that it renders easy the

cleaning of pipes, sprinklers and siphons blocked by growths, without disconnecting the system.

The Design of a Hypochlorite Disinfecting Plant. In the disinfection of sewage by the use of bleaching powder the bleach must be first dissolved so as to make a strong solution (2–5 per cent) and thoroughly mixed (usually by mechanical means). This strong solution is then generally diluted so that it has a strength of about 1 per cent and fed at a definite rate into the sewage to be treated. The latter aim is generally achieved by the use of some form of dosing tank or orifice box in which a constant head is maintained by means of a bronze float valve similar to those used in water closet flush tanks. Mixing tanks, solution tanks and dosing tanks for the use of small plants are supplied by many commercial firms in various designs.

From the dosing tank, the disinfecting solution is added to the sewage by means of gauged outlets, so that the amount added can easily be adjusted. These outlets may be connected with pipes arranged in the form of a grid, so that the disinfecting liquid comes in contact with the sewage at various points, and to bring about a thorough mixture it is well to run the sewage through a channel containing baffling boards.

The disinfection tanks, into which the sewage then passes, should be of sufficient size to contain approximately one hour's flow.

The action of the disinfectant is very rapid at first, and is then much slower, as the chlorine is used up by combination with the organic matter present. After a time, indeed, the total number of bacteria in the disinfected effluent begins to rise again as the chlorine disappears and renewed multiplication sets in. This secondary increase does not, however, of course include the pathogenic forms. The results, tabulated on p. 461 by Phelps (1909), from some of the Technology experiments, indicate that most of the disinfectant action takes place in the first ten minutes, and that a storage of one hour is ample for all practical purposes. It must be remembered that chlorine exerts a vigorous corrosive action upon metals; and this must be guarded against in the construction of this part of the plant. The tanks themselves may be of wood or masonry. The pipes and connections should be of phosphor-bronze or cast iron; and all parts should be so arranged that they can readily be taken apart for cleaning.

TABLE CXXIX

RELATION BETWEEN TIME OF CONTACT AND EFFICIENCY OF DISINFECTION WITH HYPOCHLORITE OF LIME

Available chlorine, 5 parts per million.

Boston experiments. (Phelps, 1909.)

Bacteria remaining per 1,000,000 initial.

Time of contact.	10 min.	15 min.	1 hour.	2 hours.
August 6	.....	1,100	160	150
" 9	2,500	190	58	7
" 10	10,000	270	.....	40
" 11	3,500	570	154	100
" 14	47,000	1,100	700	570
" 15	4,200	210	120	120
" 16	1,200	240	160	130
" 17	9,800	800	260	150
" 20	400,000	12,000	7000	5500
" 21	28,000	2,100	1300	1000
" 23	1,300	230	110	31
Average	50,000	1,700	950	700
Per cent	5	0.17	0.10	0.07

Fig. 130, from a design prepared by Prof. E. B. Phelps for Rahway, N. J., gives a good idea of a disinfecting plant for a small town.

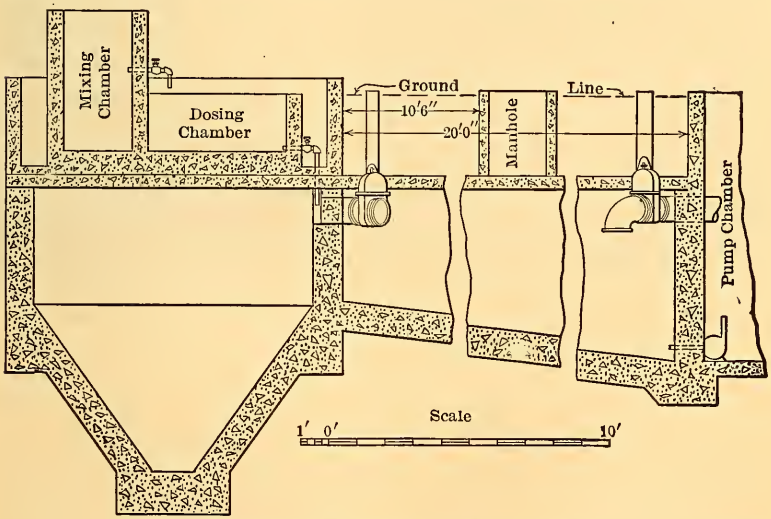


FIG. 130. Detritus Tank and Disinfecting Tank at Rahway, N. J. (courtesy of E. B. Phelps).

A good example of a hypochlorite disinfecting plant was designed to treat the sewage from a district of Philadelphia with a population of 10,000 by W. L. Stevenson (1913). It consisted of a mixing tank of white cedar, 3 feet in diameter, and 2.5 feet deep, lined with cement mortar on wire lath. The liquid was delivered from this mixing tank by a small centrifugal pump to

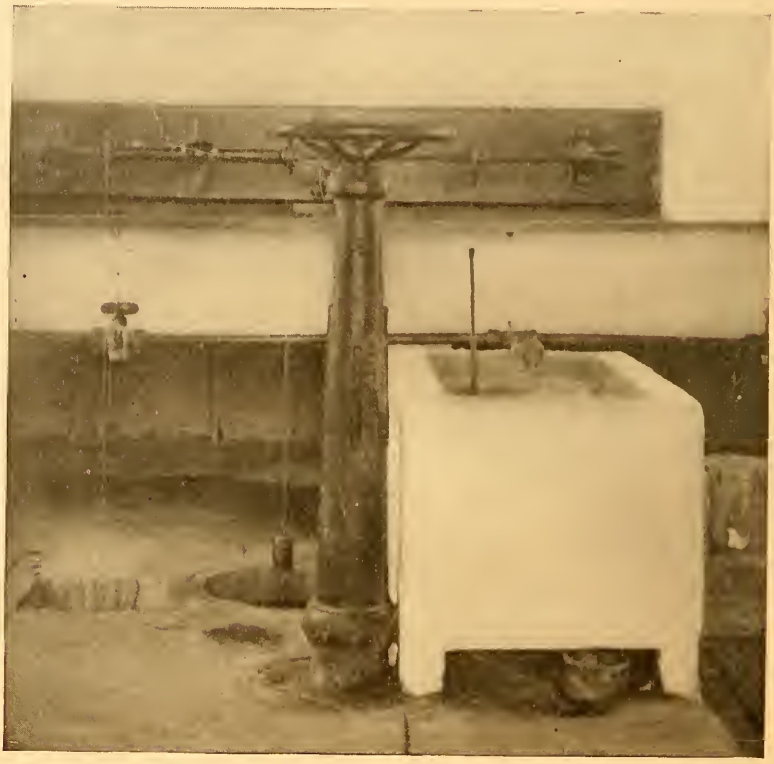


Fig. 131. Hypochlorite Disinfecting Apparatus at Atlantic City (courtesy of C. G. Wigley).

two solution tanks, each 4 feet in diameter and 6 feet deep. All tanks, pipes and valves were coated with Minwax clear waterproofing. Each tank was provided with an overflow pipe and a drain to remove the insoluble residue which collects at the bottom. A floating arm was designed to draw off the liquid from the solution tanks at a depth of 6 inches below the surface to the

constant head orifice tank. Dilution water was added to the discharge from an orifice tank.

Disinfection with Liquid Chlorine. In sewage disinfection, as in the treatment of water supplies, there has been a growing tendency in recent years to substitute liquid chlorine for hypochlorite solution as a disinfecting agent. Major C. R. Darnall, U. S. A., was probably the first to suggest this process, in 1910. The process has a number of distinct advantages. It does away with the necessity for mixing tanks, solution tanks, sludge pumps, etc. The liquid chlorine treatment requires nothing but a simple regulating device and the cylinders of liquefied chlorine gas (see Fig. 132). It eliminates the handling of hypochlorite which is a dusty and offensive process. It is far easier to regulate than the bleaching powder treatment; and while its cost was somewhat greater before the war it is now (1918) a more economical procedure.

Liquid chlorine disinfecting apparatus is made in both hand control and automatic control designs, by the Wallace and Tiernan Company, the Electro-Bleaching Gas Company and other firms; and where such an apparatus is used the only other provision that need be made is for a storage tank to hold the sewage for a period of perhaps one hour until the process of disinfection has been completed.

Practical Results of Sewage Disinfection Plants in Actual Operation. The most important application of sewage disinfection on a practical scale in this country has been made at Providence, R. I. With a view to avoiding the pollution of oyster layings the effluent from chemical precipitation and from sedimentation without chemicals was treated with 3–9 parts of available chlorine in the form of bleaching powder. When no lime was used, the costs of sedimentation and disinfection amounted to $2.85 and $2.50 for the years 1912 and 1913 respectively and the monthly average purification effected ranged from 55.1 to 99.9 in 37 degrees count and from 94.3 to 99.6 in *B. coli* count. In 1914 a return was made to the practice of adding lime to the sewage before sedimentation, since the treatment of effluent from chemical precipitation gave better results than the treatment of the effluent from plain sedimentation. Disinfection at Providence is at present (1918) accomplished by the use of chlorine gas delivered direct from a bleach factory adjoining the works.

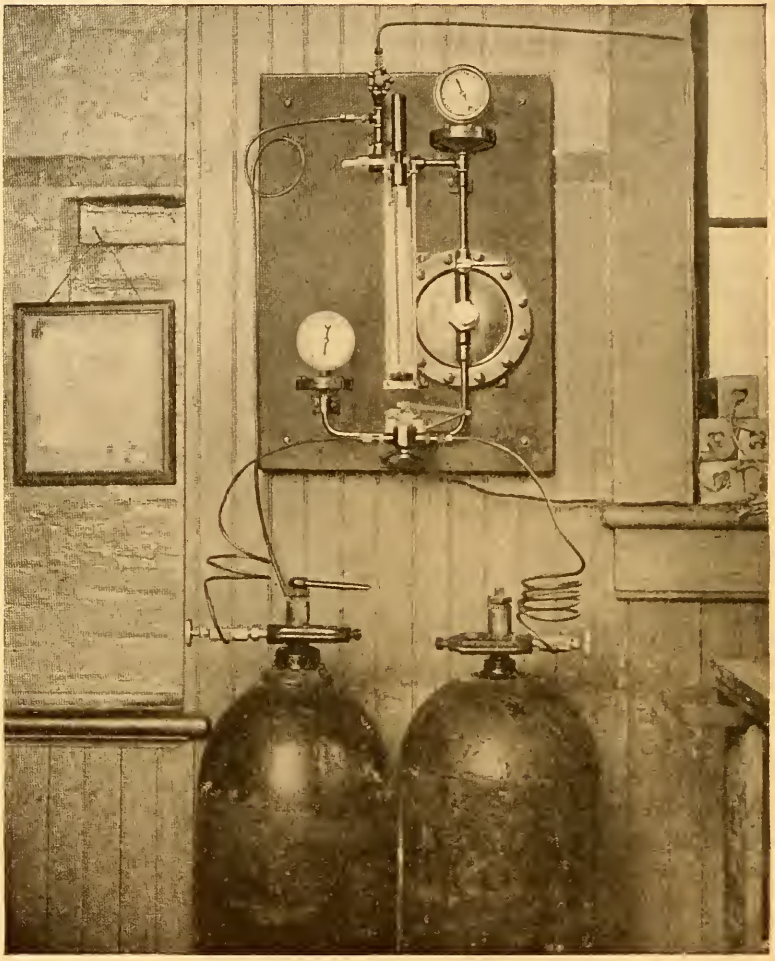


FIG. 132. Apparatus for Disinfection with Liquid Chlorine (courtesy of Wallace and Tiernan).

One of the best examples of a disinfecting plant that has been operated with care and has therefore given good results is at Mt. Kisco, N. Y. This plant was built and is operated by the city of New York for the protection of its water supply, on the drainage area of which the village of Mt. Kisco is located. The sewage, from a population of 3000 persons, is first screened and settled, then treated in double contact beds, then subjected to secondary sedimentation, then filtered through sand beds and finally disinfected with bleaching powder, the amount of available chlorine applied varying between 10 and 20 parts per million. The bacterial character of the sewage and the effluents at various stages of the process are shown in Table CXXX.

TABLE CXXX

BACTERIAL RESULTS OF SEWAGE TREATMENT AT MT. KISCO, N. Y.
(Coffin and Hale, 1916.)

	Bacteria per c.c. 37° C. count.	*B. coli.* Per cent positive tests in							
		0.000001 c.c.	0.00001 c.c.	0.0001 c.c.	0.001 c.c.	0.01 c.c.	0.1 c.c.	1 c.c.	10 c.c.
Raw sewage.....	1,480,000	37	84	98	100	100	100	100	100
Septic effluent.....	400,000	22	49	92	100	100	100	100	100
First contact effluent	560,000	20	59	92	100	100	100	100	100
Second contact effluent.....	470,000	16	59	92	100	100	100	100	100
Secondary settling basin effluent.....	460,000	14	49	90	100	100	100	100	100
Sand filter effluent..	38,000	0	10	60	83	100	100	100	100
Disinfected effluent.	38	....	....	...	...	..	4	8	39

Of course this is a very unusual disposal plant, in which no expense in construction and no care in supervision has been spared to secure ideal results. It is of great interest however as showing what excellent results may be secured when it is necessary and desirable to take all possible precautions. Hale cites figures to show that 36 per cent of the samples of the final effluent were sterile, 65 per cent contained less than 10 bacteria per c.c. (agar, 37° C.), 93 per cent less than 50 bacteria per c.c., and 97 per cent less than 100 per c.c. The effluent is at all times better from a bacteriological standpoint than the stream into which it flows.

The special importance of sewage disinfection in the maritime communities of the state of New Jersey has been alluded to above.

In 1914 F. E. Daniels stated that two dozen disinfecting plants were in operation in that state, the first to be installed in point of time being the plant at Stone Harbor which went into operation in 1909.

In order that disinfection may be successful it is essential that no large masses of suspended solids shall be present as into the interior of such masses the chlorine cannot be expected to penetrate. In the Philadelphia studies it was assumed that at least all particles larger than 1 millimeter must be removed. An efficient mixture of the chemical with the sewage is also essential.

Experience has proved that the automatic apparatus controlling various forms of chlorine disinfecting plants is very liable to get out of order and that if careful and constant supervision is not provided valves will corrode and tubes will clog. Severe cold weather introduces special difficulties and the first supply of chlorine from a new tank often contains impurities which deposit in the piping.

It must also be remembered that the purifying effect upon lake or harbor waters attained by the disinfection of the dry weather flow from large sewers may be largely nullified by storm water overflows and discharge from private sewers, shipping and other possible sources of pollution.

The Cost of Sewage Disinfection. The main element in the cost of disinfection is the amount of bleaching powder or liquid chlorine required, although of course the cost of tanks, maintenance and labor must all be taken into account.

The price of the bleaching powder will depend on the nearness of the source of supply, and the cost of tanks will depend somewhat on the organic purity of the liquid. With well-purified effluents, requiring less than five parts per million of available chlorine, the storage may with advantage be prolonged over one hour, as the chlorine will not disappear as quickly as with effluents of higher organic content. The cost of labor will depend on local conditions, and for a small plant will be proportionately higher, though in a small community the care of the plant may be combined with other town work. The cost of treatment with varying amounts of bleaching powder, from one part to fifteen parts of available chlorine, has been estimated by Professor Phelps (Phelps, 1909); and according to his figures the application of five parts of chlorine, required for the effluent of a trick-

ling filter, will cost in the neighborhood of $1.75 per million gallons. To disinfect crude sewage with fifteen parts per million of chlorine will cost about $5.00 per million gallons.

Metcalf and Eddy (1916) give the following tabular statement of the cost of chlorine at certain specified prices of bleach:

TABLE CXXXI

COST OF CHLORINE UNDER VARIOUS SPECIFIED CONDITIONS

(Metcalf and Eddy, 1916.)

Calcium hypochlorite assumed as ¼ available chlorine.

Parts available, chlorine per million.	Pounds calcium hypochlorite per million gallons.	Cost of calcium hypochlorite per million gallons sewage		
		at 1.25 cents per pound.	at 2 cents per pound.	at 5 cents per pound.
0.5	12.5	$0.16	$0.25	$0.62
1	25	0.31	0.50	1.25
5	125	1.56	2.50	6.25
10	250	3.12	5	12.50
25	625	7.81	12.50	31.25

CHAPTER XV

SOME GENERAL CONSIDERATIONS IN REGARD TO THE DESIGN AND OPERATION OF SEWAGE TREATMENT PLANTS

Relation between the Composition of Sewage and the Selection of a Sewage Treatment Process. It cannot be too strongly emphasized that there is no panacea for the troubles with which the sewage expert deals. Improvements are constantly taking place; and each new process is hailed as the ultimate solution of the problem. The intermittent filter, the trickling filter, the septic tank, the Imhoff tank, the fine screen, and the activated sludge process — we have seen each one in turn occupy the center of the stage; and we have seen each one fall back into its place as one of many procedures which may be employed with success when local conditions warrant. In the combination of various processes, old and new, to suit the particular needs of the particular problem will be found the only safe road to success. First of all the composition of the particular sewage to be treated may often prove a controlling factor which will determine whether one procedure or another may be expected to yield adequate results. Sewage, containing undecomposed masses of suspended solids will prove far more favorable for treatment by fine screens than one in which the suspended material has been finely comminuted. Fresh sewage will be much less likely to cause a local nuisance when treated on trickling beds than that which is stale and septic when it arrives at the treatment works. The cost of chemical precipitation will be materially affected by the iron content of the sewage and that of Miles acid treatment by its alkalinity and grease content. The amount of detritus present is often a governing factor in the successful working of processes of clarification. Above all the presence of special types of industrial wastes may completely upset any calculations unless their nature is well understood. Wool wastes at Clinton, Mass., and pickling liquors at Worcester, Mass., and New Britain, Conn., have for example caused serious difficulties in the operation of

intermittent filters. The stockyards sewage at Chicago offers absolutely individual problems; and the sewage at New Haven, Conn., as pointed out in an earlier chapter, contains so much colloidal copper from a munition factory as to be appreciably disinfected, and is quite unsuitable for treatment by the activated sludge method and probably by other biological processes.

The whole problem of trade wastes is one which must occupy a large share of the attention of the sewage expert in the future. Such wastes usually offer such serious difficulties in connection with the ordinary processes of sewage disposal that it would in many cases be advisable to require their treatment by some special procedure before they are permitted to enter the public sewers or to discharge into any stream. The attempt to deal with these wastes by the biological methods applicable to the treatment of domestic sewage has not generally been successful, and it seems probable that they should be studied rather from the standpoint of the industrial chemist than from that of the sanitary engineer. The action taken by the State Department of Health of Connecticut in frankly turning over a problem of this kind (the treatment of wax wastes from silk mills) to a firm of consulting chemists (the Arthur D. Little Corporation) is significant of a useful type of co-operation with experts who may perhaps succeed in the recovery of by-products from wastes which have proved most difficult to handle along ordinary sewage disposal lines.

Varying Standards of Purity Required in the Final Effluent from Sewage Treatment Plants. The second important variable in the selection of a sewage treatment process is the nature of the final effluent to be secured.

The work of the engineer has been defined as the art of doing for one dollar what any fool can do for two. It is essential, not only to solve a given problem of sewage disposal successfully, but to solve it at a minimum cost. In practice this means taking the largest possible advantage of local facilities for disposal by dilution, the one process of purification which may often be secured without cost. The amount of diluting water, its normal oxygen content, the facilities for mixture, the direction and strength of local currents and the uses to which the body of water is put (for drinking, bathing, oyster culture and the like) will all enter into the consideration of the extent to which dilution may

be relied upon for the digestion of the organic matter present. The turbid streams of the middle West can receive a cloudy effluent and even a slightly putrescible one without producing noticeably undesirable conditions such as might be created by the discharge of the same effluent into a clear stream.

It may fairly be maintained that sewage should in all cases and at all times be so disposed of that no appreciable offense shall be created from the standpoint of either sight or smell and that no sludge deposits shall accumulate in adjacent waterways.

The Metropolitan Sewerage Commission of New York, after an exhaustive study of local conditions, formulated what may be considered as an irreducible minimum standard for conditions in which the factor of dilution is relied upon to do the largest possible proportion of the work. The Commission specified "that garbage, offal, or solid matter recognizable as of sewage origin should not be visible in any of the city's waterways. Marked discoloration or turbidity due to sewage or trade wastes, effervescence, oily sleek, odor or deposits should not occur, except, perhaps, in the immediate vicinity of sewer outfalls and then only in such places and to such an extent as were permitted by the authority having jurisdiction over the sanitary condition of the harbor. The discharge of sewage should not materially add to the formation of deposits injurious to navigation. The preservation of fish life not being a consideration, it was at first recommended that the dissolved oxygen in the water be not allowed to fall below 3 cubic centimeters per liter of water. This amount of oxygen would be about 58 per cent of saturation under summer conditions. The oxygen limit was finally eliminated as not necessary, for, if the other conditions were complied with, it was evident that there would then be ample oxygen in the water."

Soper, Watson and Martin (1915) in their report on the Chicago problem suggested as a tentative list of minimum requirements that "(*a*) solids which can readily be recognized as of sewage origin should not occur in any part of the waterways of the Sanitary District; (*b*) putrefactive decomposition should not occur in those parts of the Chicago River and its tributaries and outlets which pass through built-up sections of Chicago or any other city or town or village; (*c*) conditions necessary for healthy fish life should be maintained from Lake Michigan through the South

Branch, Drainage Channel, Desplaines and Illinois Rivers and in at least the larger arms and tributaries of these waterways."

Such conditions as those outlined above can be secured under certain conditions without any preliminary treatment at all, by the direct discharge of crude sewage through submerged outlets. Where the amount of sewage is larger, screening at least will be required or in other cases tank treatment. When the proportion of sewage to diluting water becomes still greater, biological oxidation becomes essential and in the extreme case, such complete purification as is attained by the use of intermittent filters or the activated sludge process. In every particular instance means must be adjusted to ends after a careful study of local conditions from the double standpoint of efficiency and economy.

Other Local Factors Affecting the Selection of a Sewage Treatment Process. Aside from the character of the sewage and the nature of the effluent required, there are many other local factors which profoundly affect the availability of different processes of sewage treatment in a given case. Among the most important of these conditions are the area of available land, the character of the neighborhood of the disposal works, the presence of suitable filtering material, the available head for the operation of sewage disposal devices and the local cost of power. In such a lake city as Milwaukee where sewage disposal plants can only be located on the water front, where land is expensive and local nuisances must be avoided at all costs, activated sludge treatment has very special advantages (if the sludge itself be carefully handled), while at Indianapolis land for trickling filters is available in relatively unsettled neighborhoods. The possible cost of pumping enters as a most important factor into the selection of a disposal area; and where a limited head is available at a certain point without pumping this fact may be a determining element in favor of contact treatment or some other process requiring a minimum head.

The Value of Preliminary Experimental Studies in the Selection of a Sewage Treatment Process. The art of sewage treatment is a relatively new art and one that is undergoing constant and rapid development. Furthermore, as has been pointed out above, its application in a given case will be profoundly modified by variations in existing local conditions.

Experiment stations for the preliminary study of special sewage treatment problems by the testing out of various devices on a practical scale have therefore been established by many states, cities and engineering schools in the United States and have been most fruitful in their results. The dates of some of the most important of these investigations are tabulated below, data furnished by Fuller (1912) and Pearse (1917) being brought up to date.

TABLE CXXXII

AMERICAN EXPERIMENTAL STUDIES OF SEWAGE TREATMENT

Location.	Date.	Approximate cost.
Lawrence, Mass. (Mass. State Board of Health)	1887–1917	$300,000*
Worcester, Mass.	1900–1917	61,160
Mass. Institute of Technology, Boston, Mass.	1903–1911	55,000
Columbus, O.	1904–1905	44,000*
Waterbury, Conn.	1905–1907	14,000
Reading, Pa.	1906	2,500
Baltimore, Md.	1907–1908	17,500
Gloversville, N. Y.	1907–1909	13 570
Philadelphia, Pa.	1908–1910	24,500
Chicago, Ill. (Sanitary District)	1909–1917	190,100
Akron, O.	1911–1912	9,400
Cleveland, O.	1912–1916	88,000
Brooklyn, N. Y.	1912–1916	50,000
Michigan, University of	1913–1916	6,000
Decatur, Ill.	1914	5,500
U. S. Public Health Service	1914–1916	75,000
Milwaukee, Wis.	1914–1916	180,000
Illinois State Water Survey	1914–1916	10,000
Richmond, Borough of	1914	50,000
Houston, Tex.	1915–1916	5,524
New Haven, Conn.	1917–1918	18,000

* Includes also work on water purification, and in case of Lawrence routine bacteriological work of the State Board of Health.

Over a million dollars has been expended at these various testing stations; but the return has been well worth the investment. So far as general contributions to the art of sewage treatment are concerned it will be remembered that the work of the Lawrence Experiment Station of the Massachusetts State Board of Health gave us not only the process of intermittent filtration but the fundamental basis for all modern biological processes of sewage treatment. The early Worcester studies contributed materially to our knowledge of the septic process. The Massa-

chusetts Institute of Technology and Columbus experiment stations demonstrated the practical importance of the trickling filter under American conditions. The Reading investigation gave us a new and valuable type of fine screen; and the Cleveland studies threw much light on the possibilities of fine screening. The work at Brooklyn, Chicago, Cleveland, Urbana and Milwaukee has played a most important part in the development of the process of activated sludge treatment.

Fig. 133. Sewage Experiment Station, Columbus, Ohio.

In addition to the establishment of general principles of sewage treatment and the development and evaluation of new processes the testing station is of inestimable benefit in adjusting generally useful procedures to special local conditions. Thus the sewages tested at Gloversville, Chicago (Packingtown sewage) and New Haven were all so peculiar in character as a result of their content of industrial wastes that only a special investigation could determine what methods of treatment could best be applied. It is eminently desirable that special experimental

studies should precede the installation of all works sufficiently large to justify the expenditure involved.

The value of experiment stations in connection with water and sewage disposal practice and certain of the practical problems which arise in connection with the design and operation of such stations have been discussed in an illuminating manner by Pearse (1917).

Progressive Development of Sewage Disposal Problems. Even with the greatest care in regard to preliminary design it cannot be assumed that the sewage problem of a given community, when once solved, will continue to be solved for all future time. Many a plant, entirely adequate at first, has ultimately failed because no account has been taken of an increasing volume of sewage flow, so that tanks and filters have become overloaded beyond their possible capacity. Provision should always therefore be made for future expansion of sewage works with an increase in contributing population.

Another important factor which enters into the situation is the increasing perfection of the results to be obtained, likely to be demanded as a result of rising standards of public health and public decency and increasing populations in the vicinity of sewage works and along the streams or harbors into which they discharge.

A summary of the history of the sewage disposal problem at Worcester, Mass., by Butcher (1917) shows how constant the struggle must be to secure and maintain reasonably satisfactory conditions. A Report by the State Board of Health, Lunacy and Charity in 1881 emphasized the bad condition of the Blackstone River and urged sand filtration for Worcester sewage. In 1886 the city engineer after an inspection trip in Europe recommended chemical precipitation. The legislature in that year directed the city to "remove from its sewage . . . the offensive and polluting properties and substances therein, so that after its discharge into said river . . . it shall not create a nuisance or endanger the public health." Chemical precipitation was begun in 1890 and the works materially enlarged in 1893. Suit was however brought against the city in 1895 to compel more adequate compliance with the act of 1886. The plant was therefore enlarged and sewage and storm water separated. Studies of dewatering sludge led to the installation of sludge presses in 1898

and sand filter construction was begun (72.5 acres had been completed by 1915). Studies on Imhoff treatment begun in 1900, on contact beds in 1904, and on trickling beds in 1906, led to endorsement of Imhoff tank and trickling filter treatment in 1912. More recent studies however point to activated sludge as perhaps the best solution of the problem and while these elaborate studies have been carried on the State Department of Health has been urging special legislative action to hasten the relief of the population on the still polluted Blackstone River.

The experience of Chicago may also be cited as showing the temporary character of many sewage disposal projects. Wisner (1911) stated that the Chicago Drainage Canal had given good results for 12 years, but that Imhoff tanks were now necessary and that trickling filters would be needed around 1930. Four years later Soper, Watson and Martin (1915) reported that "it is wrong in principle and in practice . . . to discharge the crude sewage into the waterways" and that "The existing policies have not given the city a safe drinking water, nor eliminated the insanitary conditions in the Chicago River. They have made impossible a wholesome condition of the Desplaines and Illinois Rivers for about one hundred miles."

It will therefore be desirable for the sewage-works engineer to provide not only for the expansion of a given plant to meet an increasing sewage flow, but if possible to allow for such future modifications in the method of treatment itself as may probably be required by the tendency to increasing requirements on the part of sanitary authorities.

The Problems of Local Nuisance and of Sludge Disposal. It must never be forgotten that the production of a properly purified effluent and the consequent maintenance of desired standards in the body of water into which the effluent flows is only one of the essential aims of sewage treatment. Another and equally essential aim is the accomplishment of the results desired without the production of local nuisance. Odors of putrefaction and plagues of flies in the neighborhood of sewage treatment works have repeatedly led to serious complaints and damage suits. For the purpose of avoiding such difficulties sewage disposal plants should be located in as isolated a position as possible and the greatest care should be taken, both in design and in operation, to minimize the exposure to the atmosphere of pu-

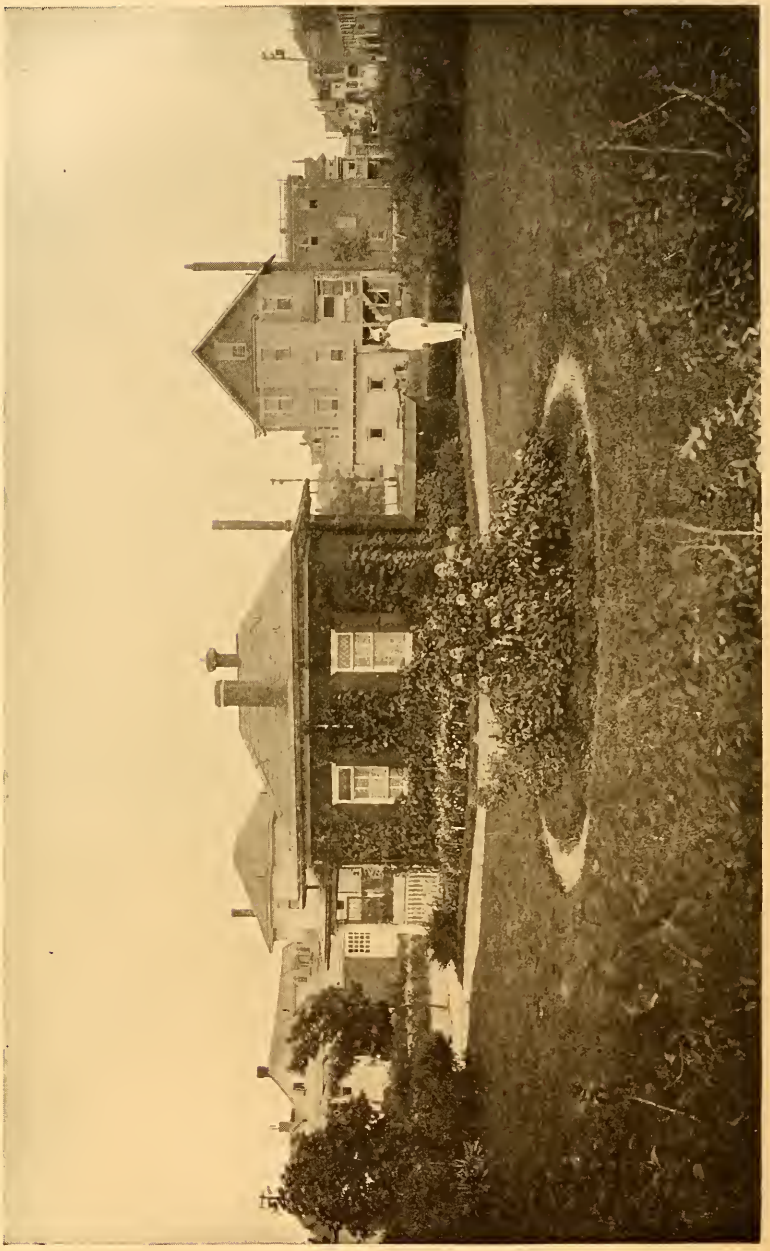


FIG. 134. An Example of a Sewage Treatment Plant made Attractive by Landscape Gardening, Raleigh Avenue, Atlantic City (courtesy of C. G. Wigley).

trescent liquid and solid materials. That this is by no means an impossible ideal is proven by experience. Daniels (1914) shows a picture of a most attractive residential street in a New Jersey town, under which a septic tank is located from which no complaints have ever been received. In many instances Imhoff tanks and rapid filters of various types have been operated without nuisance in the immediate vicinity of dwellings, as at Cheektowaga, N. Y., for example (Eng. News-Record, 1917).

The commonest fault in sewage works, from the standpoint of design, is a failure to provide adequately for the disposal of screenings and sludge. It is strange to note how often a plant is carefully and intelligently planned to separate the solid from the liquid constituents of the sewage and to purify the latter, while the sludge is left to be disposed of by some spontaneous happy thought when the occasion shall arise. One way of evading this difficulty, particularly in the case of screenings, is to assume that they may be carried away and buried at some distant point. This may indeed offer a satisfactory solution of the problem in a given case; but all the details for the disposal of screenings and sludge should be worked out with as much care and as detailed estimates of cost as are devoted to the design of tanks and filters. It must never be forgotten that screenings and sludge are the most offensive of all the materials with which the sewage works manager must deal; and that provision for their prompt and regular removal and for their ultimate disposal without nuisance forms, not an incidental feature of sewage treatment, but its most central and most difficult problem.

Recent Tendencies in the Design of Sewage Treatment Works. While it is essential to remember that the problem of sewage treatment is a local problem, yet there are certain broad tendencies of current practice to which brief reference should be made.

Of the various processes of sewage treatment which have been discussed, plain sedimentation, chemical precipitation and broad irrigation have perhaps the most limited application. Chemical treatment with acid for the recovery of grease may prove economical in certain cases, and broad irrigation is useful in the arid regions. For ordinary practice however the choice will probably lie between coarse screens, fine screens, septic tanks and Imhoff tanks for preliminary treatment and between sand filters,

contact beds, trickling filters and activated sludge for final oxidation.

Septic tanks seemed for a time to offer the most satisfactory method of clarification when coarse bar screens were insufficient. They were later superseded in favor by Imhoff tanks which enjoyed a brief period of great popularity. Then trouble was experienced at Baltimore, Md., Plainfield, N. J., and other plants from the foaming and discharge of foul gases from Imhoff tanks which has led George W. Fuller to recommend in several cases the use of fine screens as an alternative to tank treatment. At Cleveland the preliminary investigations made in 1913–14 led to the installation of a Riensch-Wurl screen for an experimental test period in 1915–1916. It proved unsatisfactory and the engineers in charge concluded in favor of Imhoff treatment and chlorination. George W. Fuller, who was called in as a consultant, dissented from this decision fearing that Imhoff tanks at the proposed site might create a nuisance, and recommended fine screening and chlorination. The Ohio State Department of Health hesitated to approve a plan which would discharge so large an amount of suspended solids into the lake and referred the whole matter to Prof. E. B. Phelps of the United States Public Health Service. H. P. Eddy at the time of writing is reviewing all the reports presented and a final decision has not yet been reached.

Screens will not of course yield anything like so complete a removal of suspended solids as tank treatment; and it can by no means be assumed that screening plants will be inoffensive unless special provision is made for daily removal and disposal of screenings, which are particularly objectionable when they decompose. The choice between the Imhoff tank and the fine screen in a given case will depend largely on the necessary removal of suspended solids, on local construction costs and on facilities for the ultimate disposal of sludge and screenings, respectively. The old one-story tank may often be better suited than the Imhoff for small plants on account of its relatively simple operation.

For the oxidation of organic materials intermittent filters are ideal for small communities where ample sand areas are available; and contact beds may be indicated on account of low head or desire for a compact and concealed plant, particularly in

small installations. For large cities, the choice will most often lie between trickling beds and activated sludge treatment, the determining factors being largely the character of effluent desired, the area of land available and the surroundings of the disposal site.

The table below prepared by Hering and Fuller (with estimates for activated sludge added by the authors) gives a fair idea of what may be expected from various processes of sewage treatment under practically ideal conditions of operation. It is assumed that sewage applied to contact, trickling and sand filters has received all necessary preliminary treatment and that the effluent from trickling filters has been subjected to secondary sedimentation.

TABLE CXXXIII

PERCENTAGE PURIFICATION EFFECTED BY VARIOUS METHODS OF SEWAGE TREATMENT

Method of treatment.	Suspended solids.	Organic matter.	Bacteria
Fine screens	*15	10	15
Sedimentation	65	30	65
Septic treatment	65	30	65
Chemical precipitation	85	50	85
Contact beds	85–90	65–70	80–85
Trickling filters	85–90	65–70	90–95
Activated sludge	95–98	75–90	90–98
Sand filters	95–99	90–98	98–99

* It is believed by the authors that the figures for screens are too high and that a 10 per cent removal is the maximum that can be expected in practice.

The Operation of Sewage Disposal Plants. Difficulties which have arisen in connection with sewage disposal plants in the past have frequently been due to carelessness in operation rather than to faulty design. Public interest in sewage treatment is generally perfunctory. Too often the only desire of the city fathers is to locate the disposal works at the most remote possible point and then forget their existence, — until complaints of nuisance or damage suits recall the subject in an unpleasant manner.

A sewage disposal plant like any other technical device requires intelligent attention and occasional expert supervision. As Dr. Thresh, the eminent English sanitarian, has said, "I have so repeatedly seen excellent works give bad results on account of

inefficient management, and very defective works give fair results on account of the efficiency of the manager, that I have come to regard the manager as being even more important than the works."

It is very largely as a result of poor management of intrinsically sound sewage disposal devices that the columns of the engineering press and the reports of State Health Departments so often contain disheartening accounts of sewage disposal failures. In a report made some years ago of an inspection trip of a group of sewage plants in the Middle West by one of the writers (Winslow, 1905) it was noted that of five sand filters only two were working quite satisfactorily while one was creating a considerable nuisance, the sewage " simply standing on the beds and overflowing into the creek, turning the whole area into a noxious swamp"; while of five contact beds only one was operating efficiently, two having their operating devices frozen up and a third having been abandoned as a nuisance.

Many similar instances of mismanagement are cited in the admirable book on The Operation of Sewage Disposal Plants by F. E. Daniels, which brings together the results of the author's careful study of New Jersey sewage works. Daniels points out the importance of constant attention to the operation of dosing devices, watchfulness as to accumulations of sludge or scum, prompt attention to clogging at the surface of filter beds, and the like. Disinfecting plants require specially careful supervision, Daniels says in regard to this point, " Although sewage disinfecting plants have been in operation in New Jersey for several years, it has been only in very recent times that an inspector from the State Board of Health could make a chance visit and find a plant working as it should. Invariably something would be found wrong and the excuse would be that it 'just happened.'"

A daily " log " should be kept in all but the smallest plants, showing in systematic form the important data in regard to plant operation. Such data will often prove invaluable in locating defects and improving methods. Daniels (1914) gives the following list of a few of the observations which should be made, while pointing out that special record forms should of course be prepared to fit each individual plant.

Sewage. — Volume, character, condition of suspended matter, collection of samples, record of analysis.

Screens. — When cleaned, amount and character of screenings removed, condition of screen-pit.

Tanks. — Tank in service, time of flow through tank, c.c. of settling solids in influent and effluent, per cent removed, sludge and scum in tank in feet and cubic yards, cubic yards of sludge and scum removed, condition of gas vents and settling chambers, collection of samples and records of analysis.

Contact beds. — Rate of dosing, cycle of operation, condition of stone, percentage of voids, rate of decrease of voids, cleaning stone, character of effluent, collection of samples and record of tests.

Sprinkling filters. — Rate of filtration, dosing cycle, collection of samples and record of tests, removal of growths.

Sand filters. — Rate of filtration, dosing cycle, distribution, care of beds, material removed, material replaced, character of effluent, collection of samples and records of tests.

Settling basins. — Hours of retention, ebullition of gas, cubic yards of sludge removed, collection of samples and records of tests.

Disinfection. — Number of pounds of disinfectant, strength of disinfectant, strength of solution, rate of flow of solution, dose in parts per million of available chlorine, pounds of bleach per million gallons of sewage, collection of samples and records of tests.

Miscellaneous. — Rainfall, temperature of atmosphere and of sewage, odors, pumping records, and a full account of all costs, including machinery, apparatus, chemicals, tools, raw materials, labor, routine expenses, repairs and improvements. Notes should be kept of all unusual occurrences and troubles. It is often impossible to diagnose a case if depending solely upon the uncertain memory of the man in charge.

Successful Sewage Disposal Not an Impractical Ideal. In spite of such failures of sewage disposal devices as have been discussed in the preceding sections, it is unreasonable to assume an attitude of despair in regard to the possibilities of sewage treatment. Strangely enough engineers who devote a large part of their lives to the design of sewage disposal plants have at times testified in court proceedings to the general effect that sewage treatments were inevitably bound to fail in practical operation and to create a more or less serious local nuisance. Such pessimism is wholly unwarranted. It is the belief of the authors that the modern art of sewage treatment makes it possible to design and to operate works which shall yield an effluent

of any desired degree of purity without giving rise to offensive conditions. Such design and operation will cost money. Yet it is by no means an ideal standard of perfection which we have in mind, but only such careful design and reasonably conscientious operation as are commonly found in other fields of municipal engineering and are by no means unknown in actual sewage treatment practice.

Laboratory control by an expert chemist is essential in a large plant; but small installations may operate very successfully in the hands of an ordinarily intelligent mechanic who should be perfectly competent to make the simple tests necessary, with occasional supervision from a consulting expert or an engineer from the State Department of Health. By no means all sewage plants have been failures in the past. The offensive ones come to our attention and the successes are unnoted. Daniels' book however is full of examples of small plants which have worked admirably after the operators had received a little expert advice. The excellent work of the Mt. Kisco plant has been discussed in the last chapter; and much less elaborate plants have often yielded very satisfactory results for periods of years.

The small Brighton plant of Rochester, N. Y., is an excellent example of successful sewage disposal. About a million gallons of sewage a day were being treated when this plant was visited by one of the authors in the summer of 1918. It was passed through $\frac{3}{4}$-inch bar screens and an Imhoff tank to trickling filter beds (one acre total area). The sewage was fresh and both tank and filters were operating to perfect satisfaction and without the slightest odor. The effluent from the trickling beds was settled in secondary humus tanks and contained less solid material and more oxygen than the stream into which it was finally discharged. The works are situated in a deep isolated valley and approached by a roadway lined with rose bushes so that the common jest about making a sewage treatment plant smell like a rose garden was here realized in actual fact.

The Committee on Sewage Works Operation and Analytical Methods of the Sanitary Engineering Section of the American Public Health Association has stated (C. S. W. O., 1916) that "With modern processes and proper operation it is possible to reduce objectionable sights and odors to such a point that only a prejudiced visitor will be dissatisfied."

CHAPTER XVI

DISPOSAL OF SEWAGE AND EXCRETAL WASTES IN THE ABSENCE OF A SEWERAGE SYSTEM

Types of House Disposal Plants. There are various methods by which the farm house, the isolated dwelling, or the institution which has no public sewerage system within reach can make shift to dispose of its waste materials in a sanitary manner. The nature of the process to be used will depend first of all on the question whether or not the house is provided with a supply of water under pressure. If so, the volume of sewage to be handled will be large and a sewer of some sort to carry it at least a short distance from the house will generally be necessary. If on the other hand there is no water supply connected with the toilet, immediate provision must be made for the excreta in the form of a pit or receptacle of some sort underneath the seat.

A second important factor which determines the procedure to be followed is the possibility of disposing of the wastes in the soil adjacent to the cesspool or closet. If the soil is suitable and there is no danger of polluting a well or other source of water supply, it is far better to dispose of excreta at their point of origin, as rehandling of "night soil" is always more or less definitely a source of danger.

We may classify the principal methods which under various circumstances have been found to give satisfaction as follows:

Disposal of Excretal Wastes

A. Where there is no pressure system of water supply in the house.

1. The Pail Closet.
2. The Pit Privy.
3. The Liquefying Tank.
4. The Chemical Closet.

B. Where there is a pressure system of water supply in the house.

5. The Tight Cesspool.
6. The Leaching Cesspool or Sanitary Sewage Tank.

7. The Septic Tank combined with subsurface irrigation.
8. More elaborate systems for large installations, such as Intermittent Filters, Contact Beds, or Trickling Beds.

The Pail Closet. The pail system of receiving feces in a movable receptacle, to be taken away and emptied at regular intervals, has been extensively used in the past in many European communities and is common in America in mining and other labor camps. Metcalf and Eddy (1915) describe a system of this sort in use on the unsewered streets of Moose Jaw, Sask. For final disposal, the material collected from an isolated house may be carried, tightly covered, to some point at a distance from the dwelling and dug into the ground. Where larger communities use this method, the collected excreta may be used in their crude condition for manure (as is almost universally the practice in China), or they may be worked into artificial fertilizer, or they may be burned in a cremator designed for the purpose.

The following sanitary essentials for a dry-closet disposal system are enumerated by Blasius (1894) in connection with a discussion of German systems of this type.

1. Pails of adequate capacity and complete impermeability.
2. Tight connection between pails and closets.
3. Constant ventilation of closet rooms and closets.
4. Regular and frequent removal of pails.
5. Hermetical closing of pails in transport.
6. A pail chamber under the closet, protected from frost and from the heat of the sun, and provided with an impermeable floor. This chamber should open from outside the house.
7. Complete cleansing and disinfection of the empty pails before they are replaced.

For an isolated house (as opposed to a camp in which regular daily removal of pails can be provided) it is very important that the receptacle should be of ample capacity so as to minimize the nuisance and danger incident to handling. Horton (1915) recommends a No. 3 ash can (15 inches in diameter and 26 inches high) of heavy galvanized metal with handles. Fig. 135 shows the arrangement of a privy equipped upon this plan as recommended. He considers that such a privy should serve for a family of six persons and would ordinarily require cleaning but once a

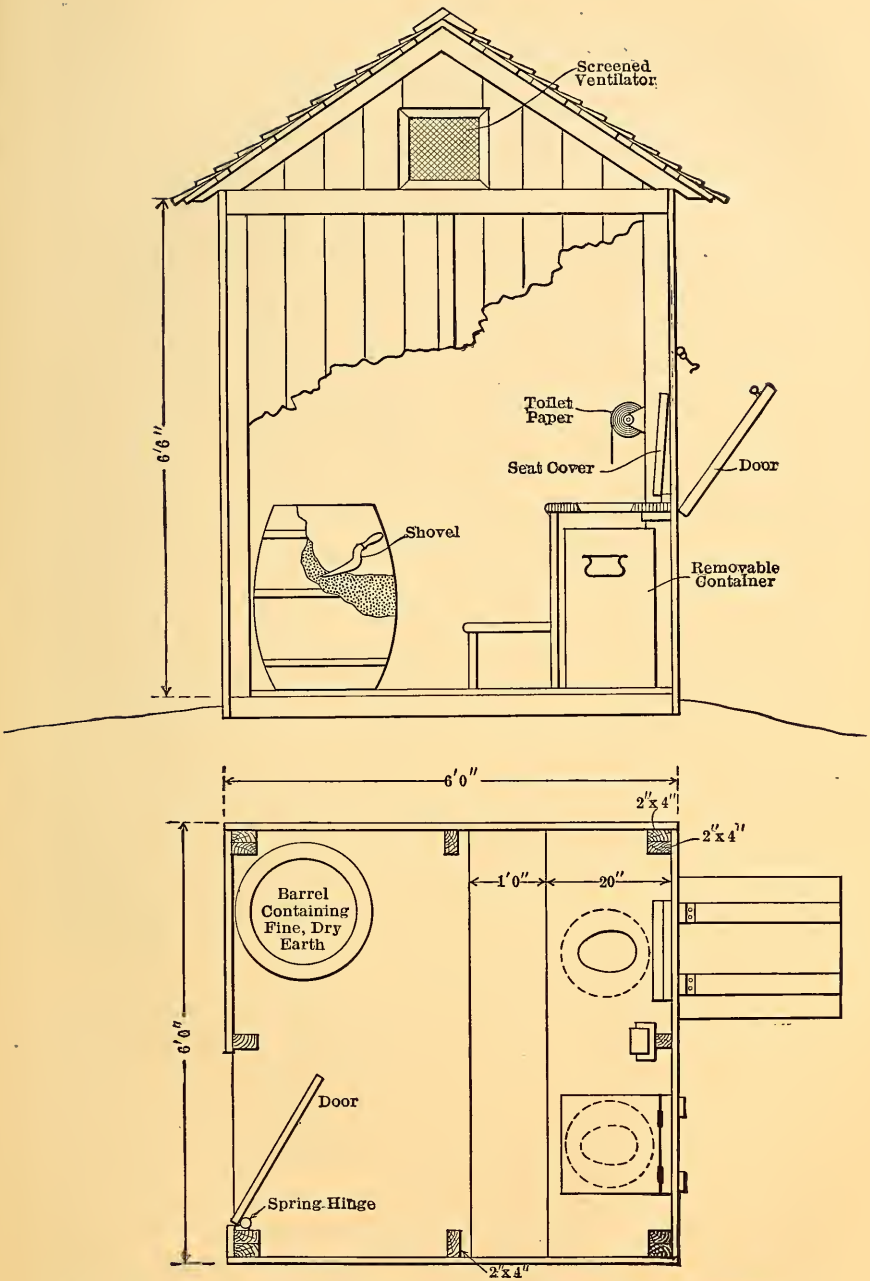


FIG. 135. A Good Type of Sanitary Privy with Removable Receptacle (courtesy of Theodore Horton).

month. A barrel of fresh dry earth or loam should be provided, with a small shovel to sprinkle it over the excreta after each use. In cold weather the outside of the privy should be protected if possible against freezing by a loose covering of leaves or straw.

In connection with this or any other type of outside privy it is essential that the house should be tightly built and all openings screened. The possibility of fly transmission of disease is after all the principal danger in such a system of disposal, and whatever other precautions are taken this one should always be insisted upon. Messer (1917) urges that the privy should be ventilated by a shaft run up from the seat through the roof.

The Pit Privy. In labor camps and other organized communities the pail system may be operated with reasonable success. For the isolated farm house however the practical difficulties in the way of securing regular removal of the pails or barrels are considerable, and there is always danger of infection from careless handling and exposure of the excrement during removal. Where there is no danger of polluting sources of drinking water through the soil it is of great advantage to avoid these difficulties and dangers by a final earth disposal of the discharges where they are deposited. For remote rural districts of the south many sanitary authorities are therefore recommending a return to an improved form of the old-fashioned pit privy.

The tightly built vault under the toilet, once common in northern rural communities, has nothing to commend it. If excreta are to be periodically removed the pail system should be used, for vaults cannot be emptied in a clean and sanitary manner. The pit privy on the other hand stands over a simple excavation in the ground, and when this excavation is two-thirds full the privy house is removed to a new hole and the first one is carefully covered with earth.

Horton's design for a privy of this type is reproduced in Fig. 136. The pit as will be noted is shored with planks and braced to prevent caving in. The earth about the top should be sloped away or a small trench dug around the site to deflect surface water during rains. Such a privy may give very satisfactory results if the house is tightly constructed so as to exclude flies, and if the location of the pit and the character of the soil is such as to avoid any danger of polluting a water supply. Its cheapness and ease of operation are the strong arguments in its favor.

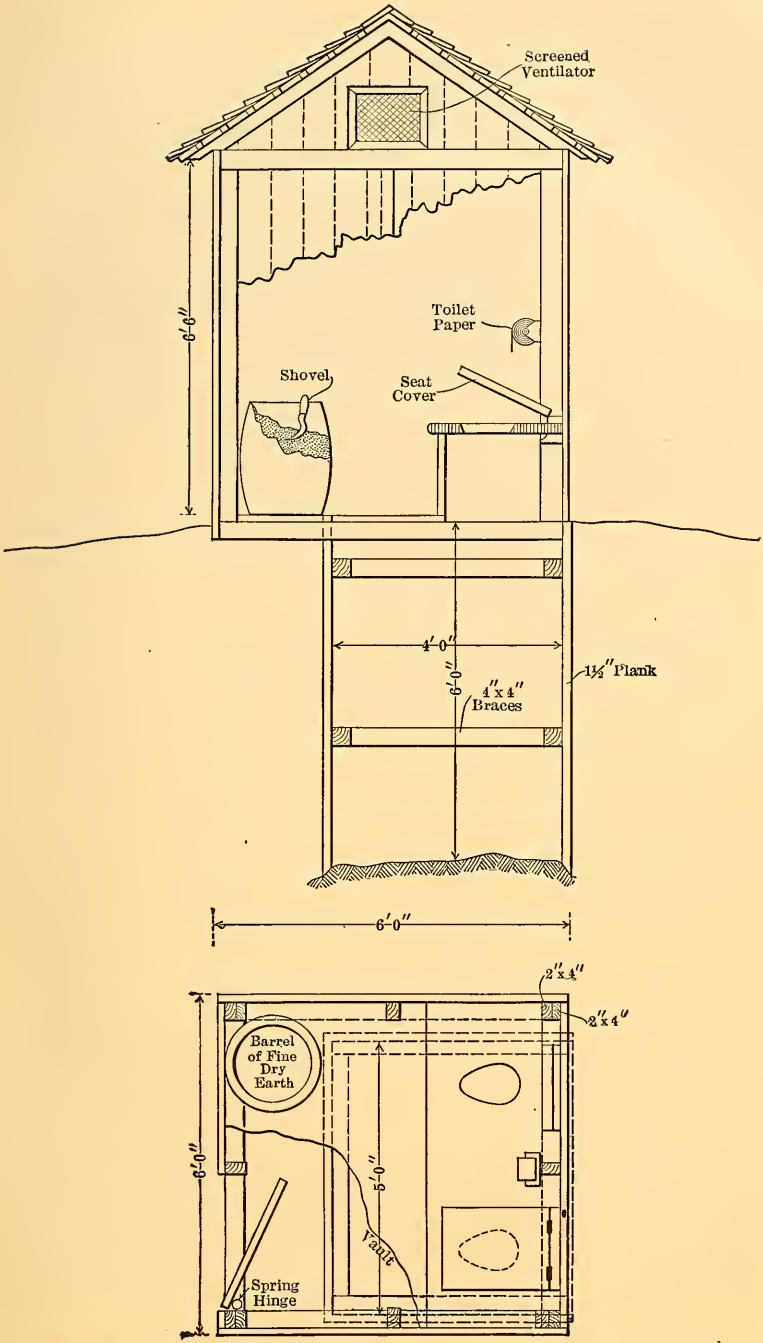


Fig. 136. New York State Health Department Design for Pit Privy (courtesy of Theodore Horton).

The Liquefying Tank. A distinct improvement on the pit privy, where the most rigid economy is not essential, is the liquefying tank originally devised by Lumsden, Roberts, and Stiles of the U. S. Public Health Service (Lumsden, Roberts and

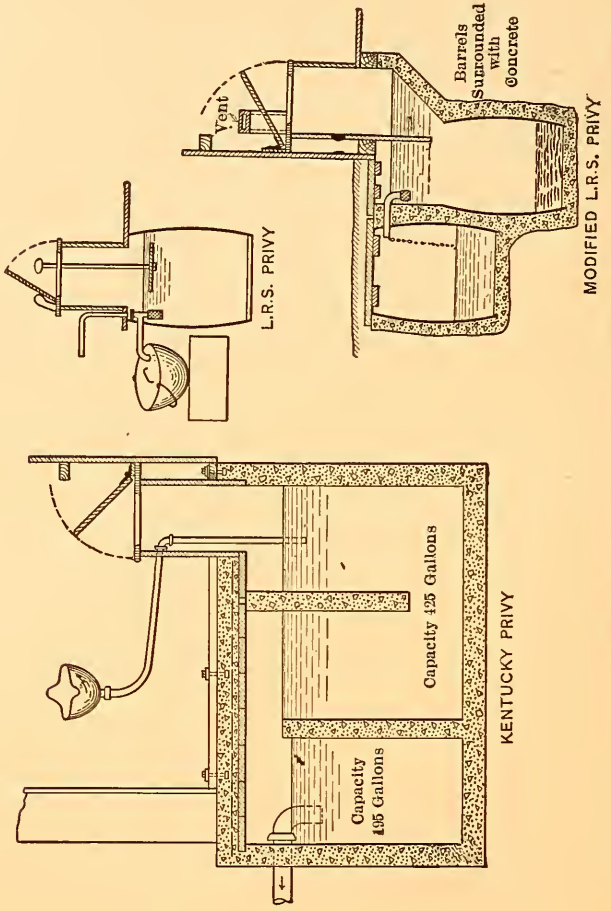


Fig. 137. Structural Details of Various Sanitary Privies (courtesy of Richard Messer).

Stiles, 1910), and called, from its inventors, the L. R. S. privy. The excreta in this closet are received in a tight chamber filled with water and provided with an overflow for surplus liquid, discharging into a removable vessel or directly into the ground. The storage of the excreta under water prevents fly breeding,

reduces odors to a minimum, and leads to a liquefaction of solid matters on the principle of the septic tank, leaving only a comparatively inoffensive liquid effluent to be removed or disposed of in the soil. In warm climates evaporation may almost balance the urine added so that for considerable periods the apparatus may not require any attention.

Fig. 137 shows the original L. R. S. privy in which a barrel is used for the liquefying tank and a kettle for the overflow, a modification suggested by Messer (1917) in which barrel receptacles are embedded in concrete, and a more elaborate concrete chamber advocated by the Kentucky State Board of Health. The latter is practically a small septic tank under the closet and is supposed to discharge its effluent into a subsurface irrigation system. In the first two types the outflow to the second receptacle is protected by a screen and the L. R. S. figure shows an anti-splashing device, which will not be needed if the water level is two feet or more below the seat. These tanks are not unlike the combined latrines and septic tanks in use in tropical countries (Clemesha, 1910).

The objection to all these devices in very poor rural communities is their cost. Messer (1917) estimates the cost of a privy house for a one-seated privy at $12.50, and the additional costs for the rest of the equipment as follows:

	Labor.	Materials.	Total.
Pit privy	$0.50	$1.50	$2.00
Pail closet	1.00	1.50	2.50
L. R. S. privy	3.00	2.00	5.00
L. R. S. privy (Messer modification)	6.00	5.50	11.50
Kentucky privy	20.00	12.00	32.00

Chemical Closets. A still more elaborate and costly arrangement is the closet provided with a galvanized or metal receptacle containing a chemical solution which deodorizes and disinfects the excreta as they are discharged. Such closets are usually designed to be located inside a dwelling or school-house and are often ventilated by a duct connected with the chimney or a special stack. These chemical closets are all patented and of various types. Some of them are highly efficient from a sanitary standpoint but all are rather costly.

The Cesspool. With a pressure water supply in the house all of the above devices should be superseded by a sewer which will carry the diluted waste materials away from the dwelling to a conveniently chosen point of disposal. If the house be situated near a large body of water, which is not used for water supply, bathing or any other similar purpose, it may be proper to dispose of the sewage by dilution according to the principles described in Chapter II. Where this is not the case the cesspool is the usual device adopted.

The cesspool may be of two general types, tight or leaching. Tight cesspools, like tight privy vaults, require the periodic removal of the contents. If neglected, as is frequently the case, they may overflow on the surface of the ground, and the process of emptying is often a source of nuisance and danger. In certain communities where this process is in use special tank carts are provided, into which the contents of the cesspools are pumped by diaphragm pumps without exposure to the air (see p. 5).

If there is no danger of water pollution it is more economical, and more sanitary as well, to use the cesspool for the reduction of the solid portions of the sewage by septic action and to provide for the escape of the liquid effluent into the soil. The simplest way to do this is by building a leaching cesspool, usually a simple dry-laid masonry well with a dirt bottom. When the cesspool becomes filled with solid matter a new one must be dug and connected. The masonry should be drawn in near the top of the well and provided with a heavy stone slab or strong oak cover or an iron manhole cover raised above the surrounding surface of the ground. If the soil be porous, and the well not near by, such a device may remain in operation for a long period without objectionable results.

Metcalf and Eddy (1915) describe a modification of this plan devised by Prof. Robert Fletcher and extensively used under the auspices of the New Hampshire State Board of Health. It is called a Sanitary Sewage Tank, and consists of a cesspool provided with an overflow consisting of an open-jointed pipe laid in a trench filled with cinders over cobble stones.

Septic Tank and Subsurface Irrigation. Exactly the same principle may be applied on a more elaborate scale for large residences or institutions by the installation of a septic tank and subsurface irrigation or absorption system. Ogden and Cleve-

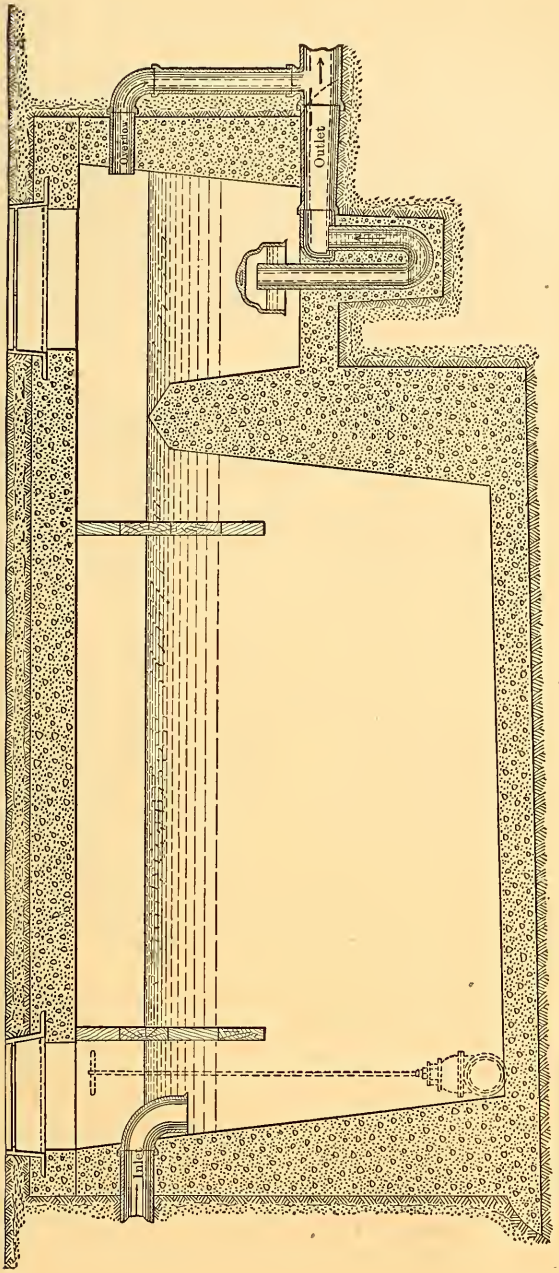


FIG. 138. Septic Tank for a Large Dwelling or an Institution (courtesy of Ogden and Cleveland).

land (1913) have given an admirable detailed description of the principles which should underlie the design of plants of this type. They estimate that the tank itself should have a capacity of from five to fifteen cubic feet per person and recommend that it be built five to eight feet deep and that the width be made from one-third to one-half the length. The sewage should be discharged by an elbow below the surface and two scum boards should generally be provided. The sludge which remains undigested must be removed from one to four times a year.

In order to provide for intermittent dosage, which is essential to success, a siphon tank must be provided which may conven-

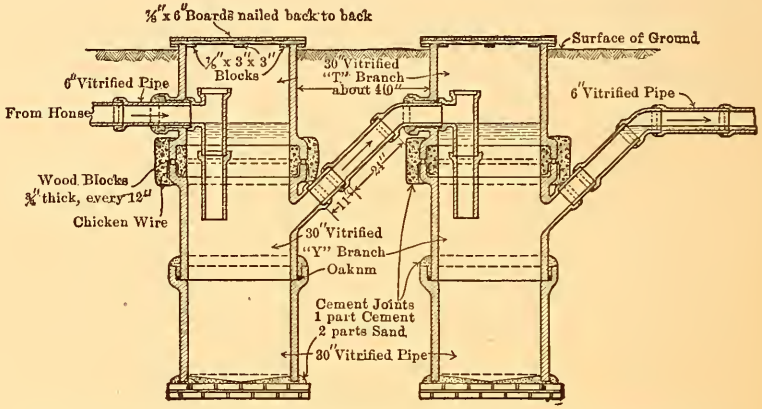

⅛" x 6" Boards nailed back to back
Surface of Ground
⅜" x 3" x 3" Blocks
30" Vitrified "T" Branch about 40"
6" Vitrified Pipe
From House
6" Vitrified Pipe
Wood Blocks ¾" thick, every 12"
Chicken Wire
30" Vitrified "Y" Branch
Oaknm
Cement Joints 1 part Cement 2 parts Sand
30" Vitrified Pipe

FIG. 139. Septic Tank Installation Constructed of Large Sewer Pipe.

iently be built in structural connection with the septic tank itself, as shown in Fig. 138. The siphon chamber is usually arranged so as to divide the daily flow into from one to three doses, or discharges from the siphon.

Another type of septic tank, which has recently been developed by the Sewer Pipe Manufacturers Association of Akron, Ohio, for use in their campaign for farm sanitation, makes use of large sewer pipe to form the tank walls. The tank is built of standard 30-inch pipe, Y and T branches, placed vertically, and is divided into two compartments connected by small pipe of standard design, as shown in Fig. 139. The pipes are set on a brick or concrete base and the upper length is placed with bell downward, requiring a special joint but permitting connection to the house

sewer without cutting the large pipe. On account of its simplicity of design this type of tank can be easily constructed by the farmer or small house owner. Also, by adding another compartment, or section of pipes, the capacity of the tank can be increased without disturbing the original construction. This tank can be followed by a siphon chamber and a subsurface irrigation system as with any other type of tank. (Fig. 140.)

Subsurface irrigation will serve very satisfactorily to dispose of the effluent periodically discharged from the septic tank, pro-

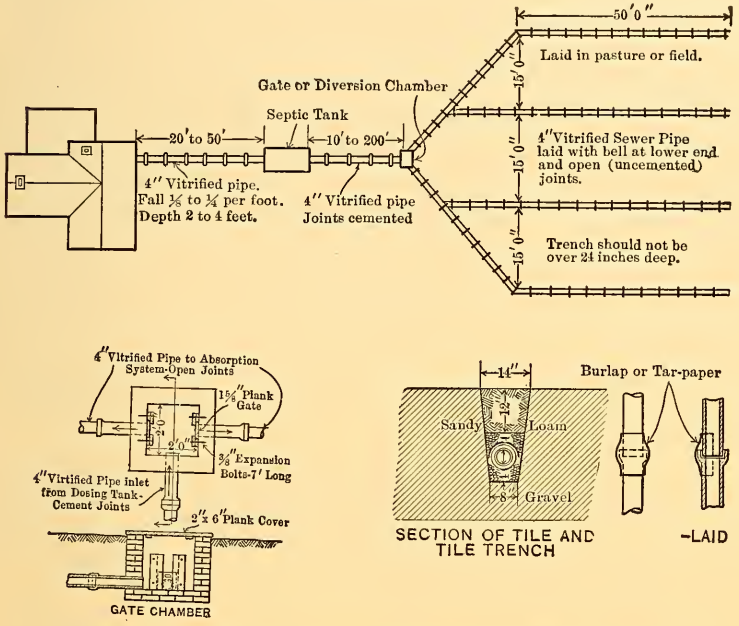


Fig. 140. Siphon Chamber and Subsurface Irrigation System.

vided the soil be of loose sand or gravel. Even in a heavy soil the results may be fairly satisfactory if a sufficient length of subsurface tiling be provided. Ogden and Cleveland (1913) estimate that 35 linear feet of 3-inch tiling per person should be provided under the former condition and 75 under the latter. It is desirable that the tile system should be divided into two or more sections so that the sewage may be diverted from one to the other to allow for a rest period. A convenient device for accomplishing this is a diverting manhole, provided with stop planks.

Main carriers should be of vitrified tile pipe with cemented joints. For a 5-inch siphon Ogden and Cleveland (1913) recommend an 8-inch main carrier with a fall of at least 6 inches per 100 feet, for a 3-inch siphon a 6-inch carrier with a fall of 12 inches per 100 feet. Distributing branch lines of 3-inch tile should be laid four to six feet apart, and 12–15 inches below the surface of the ground, with a quarter-inch space between each two sections of pipe and a piece of tar paper or a half-collar of larger diameter pipe placed over the joint to prevent clogging with earth. The slope of the distributors should not be more than 2 to 4 inches in 100 feet.

Disposal Systems for Larger Isolated Plants. For very large residences and institutions it may sometimes be better to install more elaborate disposal systems, usually including a septic tank, followed by treatment on intermittent sand filters or in contact or trickling beds, though treatment of the last two named types is not common. The marked irregularity of flow and the difficulty in securing adequate supervision are factors which must be borne in mind in designing such plants. For small plants the contact bed has the distinct advantages (in comparison with the trickling bed) of operating under a restricted head and being easily arranged so as to be out of sight; but the very fact that it is out of sight is a temptation to neglect which may lead to disaster.

The Imhoff type of tank is sometimes used in the design of residential plants; but it is doubtful whether with such small installations there is any distinct advantage in the two story feature, and the construction of such a tank is complicated.

CHAPTER XVII

METHODS OF TESTING SEWAGE AND SEWAGE EFFLUENTS

General Features of Sewage Analysis. The strength and general character of a sewage, or of a sewage effluent, depend on the amount and nature of the dissolved and suspended solid matter it contains and on its bacterial content.

The efficiency of any process of sewage treatment depends not only on the difference in the amount of mineral and organic matter before and after treatment, but on the changes that the organic matter has undergone and on the diminution in the number of bacteria that has taken place.

A complete chemical analysis of a sewage or a sewage effluent would include the following determinations:

Total solids	Total, Fixed, Volatile.
Suspended solids	Total, Fixed, Volatile.
Dissolved solids	Total, Fixed, Volatile.
Nitrogen as	Ammonia nitrogen, Albuminoid nitrogen, Albuminoid nitrogen in filtered solution, Organic nitrogen, Organic nitrogen in filtered solution, Nitrite nitrogen, Nitrate nitrogen.
Total oxygen consumed and oxygen consumed in filtered solution	3 minutes at 80° F., 4 hours at 80 F. or 2–10 minutes at 212° F.

Oxygen dissolved.
Combined chlorine.
Fats.
Acidity or alkalinity — expressed as calcium carbonate.

A complete bacterial examination should include the determination of the number of bacteria capable of developing on agar

at 20° C., the number capable of developing on litmus-lactose-agar at 37° C. and the approximate number of colon bacilli.

The primary object of sewage purification is the prevention of offensive decompositions in the body of water into which the effluent is finally discharged. The study of putrescible organic matter by the nitrogen and oxygen consumed determinations and by special putrescibility tests constitutes, therefore, the most important problem in sewage analysis. Estimations of suspended solids are also of great importance, since the deposition of decomposable matter may often cause a nuisance, even when the original effluent as a whole is stable; and in studying processes of preliminary treatment this determination is of prime importance. Bacterial examinations are in general required only in special cases where shellfish beds, bathing beaches or water supplies are directly menaced.

Sewage Sampling. The value of the data obtained from the analyses of sewage and sewage effluents depends upon the method employed for obtaining the sample for analysis. The sample to be analyzed should be a representative one, and should contain a true average amount of the various substances occurring in the sewage or sewage effluent.

Sewage and sewage effluents vary from hour to hour, both in volume and quality, and the analysis of a casual sample, or a number of casual samples, gives no trustworthy information. Day samples are often 25–50 per cent stronger than the average for the twenty-four hours. Fig. 141 indicates the actual course of the daily variations in Boston sewage (Winslow and Phelps, 1905). The chlorine curve in this case is influenced chiefly by the tides, since a considerable proportion of sea water backs up into the intercepting sewers through leaking tide gates; and the effect of a slight shower is manifest between seven and eight in the evening; but the other data indicate a fairly normal relation for the sewage of a large city in dry weather.

To obtain a true representative sample, hourly or half-hourly samples must be taken throughout a given period and mixed together in proportion to the corresponding flow.

At sewage experiment stations and at certain large plants the hourly or half-hourly collection of samples constitutes a part of the regular routine work, but at a majority of sewage plants this is not done on account of the labor required; and the strength

of the sewage and sewage effluent is determined from analyses of samples collected during a comparatively short period of time. When this is the method, the collection of samples should be

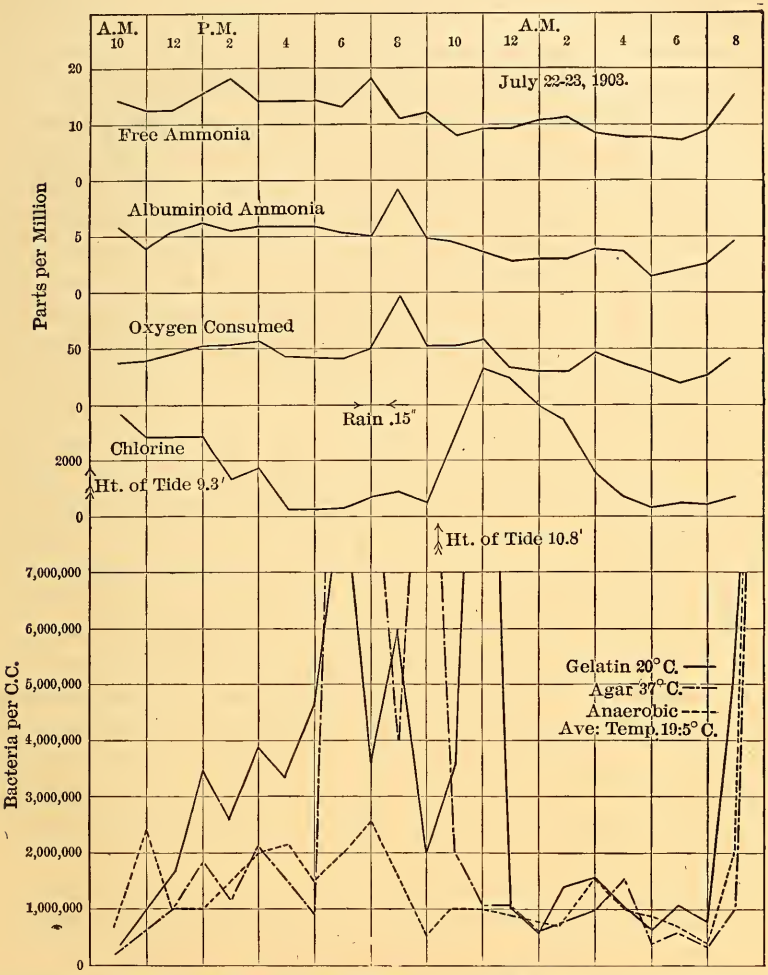


FIG. 141. Diagram of Hourly Variations in Composition of Boston Sewage.

made in dry weather and extended through seven days, though in case of small works the period may possibly be reduced to two or three days, which should not include Saturday, Sunday or Monday. The rainfall for the preceding seven days and during

the period of collection should be noted. In order to avoid multiplication of analyses, each half-hourly or hourly sample is placed in a one or two liter glass-stoppered bottle on which is noted the rate of flow and time of collection, and at the end of each twelve or twenty-four hours, a composite sample is made, the amount of each separate sample taken being determined by the rate of flow noted on the bottle. The separate samples before being mixed should be kept in an ice box, and if the composite sample is not analyzed at once it should be preserved by the addition of 10 c.c. of chloroform to 4 liters of sewage. Even with the chloroform it is essential that the composite sample should be kept at a low temperature (Lederer and Hommon, 1910); and separate samples not chloroformed must of course be taken for bacteriological examination. The method adopted at the Columbus Experiment Station as given by Johnson (1905) illustrates the proper method of obtaining a representative sample where the flow is constant:

"The samples of the crude sewage for chemical analysis were collected from the screen chamber after screening, as before stated, at half-hourly intervals throughout the twenty-four hours. These half-hourly portions were collected in four-ounce bottles, which were completely filled with sewage, tightly stoppered and placed at once in an ice chest at a temperature of about 10° C. As a midnight to midnight period was adopted as the station's official day, the first portion of crude sewage for the day was collected at 12.30 A.M., and the last and 48th portion at 12.00 P.M. At 8.00 A.M. on the day following, or 32 hours after the collection of the first portion, the contents of the 48 small bottles were mixed in equal proportion in a large glass bottle and the chemical analysis of the composite sample was immediately begun. . . .

"Samples for bacterial analysis were collected in duplicate in sterilized wide-mouthed glass-stoppered bottles at the same points as were those for chemical analysis. On account of the material changes in the bacterial content of the sewage which would take place in the bottles when stored, it was found to be inadvisable to store the bacterial samples for periods of more than about one hour. Except for occasional series of hourly samples, collected in order to ascertain the variations in the bacterial content of the sewage throughout the twenty-four

hours of the day, all bacterial samples of the crude sewage were collected at 7.00 A.M., 11.00 A.M. and 5.00 P.M. These times roughly covered the weakest, strongest and medium strength sewage of the day and enabled representative average results to be obtained. All bacterial samples were plated as soon as collected."

The methods of sampling so far described apply particularly to cases where the flow is continuous and where the efficiency of the whole plant is in question. To test the efficiency of certain portions of the process where the flow can be regulated, as, for instance, the working of a contact bed or an intermittent filtration bed which is receiving clarified sewage, very fair representative samples can be obtained by mixing equal proportions of three or four samples taken of the liquid as it enters the bed and of the liquid as it leaves the bed. In order that the samples taken at the inlet and outlet should be comparable, it is evident that account must be taken of the time the liquid remains in the bed. Whatever precautions are taken there will always be wide variations between duplicate samples of raw sewage.

Determination of Suspended Solids. One of the first, and one of the most important, points to be considered in the examination of sewage and sewage effluents is the amount of suspended solids present and the proportion of these solids present which are of organic nature. Organic suspended materials are of far more significance, from the standpoint of interference with the action of filters and on account of their liability to deposit and form foul sludge deposits in disposal by dilution, than are similar materials in solution.

Total solids in sewage are ordinarily determined by evaporating, drying at 103° C. and weighing; while the inorganic portion of the total solids is obtained by weighing the same sample after ignition. The suspended fraction of the total solids is obtained by filtering through a Gooch crucible.

A determination which has proved of special value in sewage works control is the estimation of settleable solids by an approximate volumetric method. Since a portion of the suspended matter is in such a fine state of division that it cannot be removed by any process of sedimentation, and since this fine material is of little or no significance as a contributing factor in sludge deposits, the proportion of potentially settleable solids actually

removed is perhaps the fairest measure of the efficiency of a sedimentation tank treating a given sewage.

For the determination of settleable solids a measured portion of the sewage or effluent to be tested is placed in a conical glass about 17 inches long and 4 inches in diameter at the top, holding 1 liter, with the pointed base graduated in fractions of a cubic centimeter. The sewage is allowed to settle in the glass for a standard period (commonly 2 hours) and the depth of the deposited material at the bottom is read off on the scale. The method is obviously only a rough one, but has a distinct value in the practical control of sewage works. It may be remembered however that the most putrescible materials are often those which are in so fine a state of division that they do not fall within the class of settleable solids.

The Determination of Various Forms of Nitrogen. The amount and character of the organic matter present in sewage and sewage effluents is measured chemically in two general ways, by a determination of the nitrogenous constituents, and by the determination of the oxygen taken up from a strong solution of potassium permanganate under standard conditions of time and temperature (aside from the more important biochemical tests showing the avidity for oxygen).

Ammonia nitrogen (called free ammonia in the earlier literature) is determined by diluting with ammonia-free water (if necessary), precipitating with copper sulphate and sodium hydroxide, Nesslerizing the supernatant liquid and comparing with standard solutions.* This determination is useful in giving an idea as to the amount of nitrogenous matter that has been decomposed and is present in a highly unstable form, and offers an excellent measure of the degree of purification effected by oxidizing devices, since any important amount of ammonia nitrogen indicates incomplete oxidation.

Formerly, the determination of ammonia nitrogen was supplemented by that of albuminoid nitrogen (in the past commonly known as albuminoid ammonia). This is the fraction of the organic nitrogen which is given off on treatment with alkaline permanganate solution. In recent years there has been a ten-

* No attempt will be made to discuss the details of analytical procedure which will be found in the Standard Methods for the Examination of Water and Sewage, 1917 edition, published by the American Public Health Association, 126 Massachusetts Ave., Boston.

dency to replace this procedure by the determination of total organic nitrogen by digesting with sulphuric acid. Some authorities believe that it is a mistake to abandon the determination of albuminoid nitrogen, on the ground that albuminoid nitrogen, though giving little information regarding the total amount of nitrogenous matter, does give an idea as to the amount of nitrogenous matter that is most easily decomposed. Whether this is true is somewhat uncertain; but on account of the great number of analyses of sewage where this factor has been determined, it serves as a useful guide in the classification of sewages.

In discussing the relative amounts of these various nitrogenous fractions which are likely to be present, Fuller (1912) cites the figures tabulated below from the early Lawrence studies which show how, as a sample of sewage is allowed gradually to undergo putrefactive decomposition, the total organic nitrogen decreases and the ammonia nitrogen increases, the ratio of albuminoid nitrogen to total nitrogen progressively increasing as the septic changes proceed. Other data cited by Fuller illustrate the fact that while the albuminoid nitrogen in both sewages and effluents is usually between 20 and 40 per cent of the total organic nitrogen, the ammonia nitrogen varies from a value in excess of the total organic nitrogen in stale sewage down to less than 10 per cent of the total organic nitrogen value in clarified effluents.

TABLE CXXXIV

VARIATIONS IN DIFFERENT FORMS OF NITROGEN AS FRESH SEWAGE DECOMPOSES

(Fuller, 1912.)

Age of sample, hours.	Organic N.	Percentage of Total Nitrogen as		Nitrites and nitrates.
		Albuminoid N.	Ammonia N.	
0	64	13	30	6
2	63	13	32	5
4.5	62	13	33	5
7.5	59	13	37	4
21.5	37	14	62	0.4
25.5	32	15	67	0.3
30.5	35	13	66	0
48	32	13	68	0
72	30	14	74	0
96	31	13	72	0
120	29	13	72	0
168	25	12	75	0
192	24	12	76	0

The Determination of Oxygen Consumed. Authorities in America generally consider that more valuable information can be obtained from the study of the nitrogen data than from any of the other factors, while in England and Germany greater importance is placed on the oxgyen consumed. If total carbon could be determined easily, this determination would probably be most valuable. Oxygen consumed, however, does not bear a constant relation to total carbon and is open to the same objections as albuminoid nitrogen. This factor has been used in England as a very rough guide for the classification of sewage, "strong sewage" being considered as one which absorbs from a very strong solution of potassium permanganate in four hours at 80° F. 170–250 parts of oxygen per million, "average sewage," one which absorbs 110–120 parts and "weak sewage," 70–80 parts.

The oxygen consumed does not give the total amount of oxygen necessary to oxidize completely all the organic matter present, for which no accurate method of determination is known, but only the amount given up by potassium permanganate to sewage or effluent under certain conditions.

In America the oxygen consumed has in the past been determined by adding sulphuric acid and a solution of potassium permanganate to the liquid and heating to 212° F. for two, five or ten minutes. More recently digestion for 30 minutes at the boiling temperatures has been generally adopted and is recommended as the standard procedure in the 1917 Report of the Committee on Standard Methods of the American Public Health Association. In England the temperature at which the liquid is allowed to stand is usually 80° F., and the results are observed after three minutes and again after four hours.

The data obtained by these various methods of procedure are very variable and give rise to much confusion in making deductions from results accumulated at various places. Attempts have been made to obtain factors which could be used for converting to an equivalent basis the results obtained by the different methods, and the data in Table CXXXV prepared by Fuller (1903), based on results given in Blair's "Organic Analyses of Waters" and on data supplied by Kinnicutt, are of some assistance in making comparisons, though they should be considered only as rough approximations.

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TABLE CXXXV

APPROXIMATE COMPARISON OF AVERAGE AMOUNTS OF OXYGEN CONSUMED BY SEWAGE AND SEWAGE EFFLUENTS AS SHOWN BY DIFFERENT METHODS

Method.	Temperature of solution.	Period of contact.	Relative results.
Kübel, as practised at Boston and generally in America	Boiling	5 minutes	1
Kübel, as practised at Lawrence Mass.	Boiling	2 minutes	0.65
Kübel, as practised in Germany *	Boiling	10 minutes	1.25
English official tests	80° F.	3 minutes	0.20
	80° F.	15 minutes	0.35
	80° F.	4 hours	0.60

* German results generally refer to " permanganate consumed " and should be divided by four to give oxygen consumed.

Fuller (1912) cites data showing that values obtained by two-minute boiling varied from 41 to 77 per cent of those obtained by the 30-minute method and for 36 samples averaged 55 per cent.

The English methods of determining oxygen consumed in three minutes and four hours throw light on the ratio between the more easily oxidizable substances, such as hydrogen sulphide, nitrites, urea, ferrous salts, etc., and those compounds which are not so easily acted upon by oxygen; and consequently they give more information than the American procedure.

In addition to the chemical data cited above the determination of chlorine in chlorides is of considerable importance in the analysis of domestic and residential sewage, as the chlorine in this case is chiefly derived from excreta and household waste. In sewage containing any large amount of trade waste it has only a local value as indicating whether or not the samples of sewage and effluent taken for analysis correspond.

To the above list, the determination of the amount of fat a sewage contains might well be added, for the information thereby obtained may be of importance in the consideration of methods of treatment, and, when studied in connection with the nitrogen data and oxygen consumed, gives some indication of the amount of cellulose present in the sewage.

Tests for the Stability of Sewage and Sewage Effluents. Aside from the question of suspended solids, the most important thing that we want to know about a sewage effluent is whether it

is putrescible, and if so, to what extent; or, in other words, what demand it will make upon the oxidizing capacity of the water into which it is to be discharged. This depends upon several factors, primarily upon the amount and nature of the contained organic matter, that is, the avidity of the effluent for oxygen, and upon the amount of oxygen contained in the effluent which is available to meet this demand, either as dissolved oxygen or in the form of easily reducible bodies such as nitrates. There is no chemical test which directly measures the balance between these two factors and we are therefore obliged to content ourselves with the information given by indirect methods. Among the earliest of these indirect methods is the so-called Manchester test. This test consists in determining the amount of oxygen absorbed from potassium permanganate in three minutes at room temperature, and the amount that is absorbed in three minutes at room temperature after the sample has been kept in a tightly closed bottle in the incubator at 98° F. for seven days. By incubation the organic compounds which may still be contained in the effluent are broken down by the bacteria into simpler products, and if the effluent contains sufficient oxygen to be nonputrescible these simpler compounds are completely oxidized, while if it does not contain sufficient oxygen they remain more or less unchanged, and absorb oxygen very quickly when potassium permanganate solution is added to the incubated effluent. Consequently, if the amount of oxygen absorbed from potassium permanganate in three minutes after incubation is greater in amount than that absorbed before incubation, the effluent is considered deficient in oxygen and consequently putrescible.

The Manchester method for determining the putrescibility of effluents is considered by many authorities as unreliable, and there is little question that certain effluents which remain sweet and develop no odor take up more oxygen from potassium permanganate after, than before, incubation. On this account it is held that more satisfactory information as to putrescibility can be obtained from the determination of dissolved oxygen.

Determination of Biochemical Oxygen Demand. Since the dilution finally required for raw sewage or a sewage effluent is governed by the necessary residual dissolved oxygen in the diluting water, it would seem that the most logical way to determine

how much dilution a given effluent requires would be to make various dilutions of the sample with water containing a known quantity of dissolved oxygen, and to determine dissolved oxygen in the mixture after the organic matter and bacteria had taken up all the oxygen required to satisfy them. The English sewage chemists long since reached this conclusion and developed the "English Incubation Test," in which various dilutions are made in completely filled bottles, re-aeration being prevented by suitable stoppers. After incubation for a reasonable length of time at a constant temperature, dissolved oxygen is determined. The amount of oxygen reduced, divided by the fractional percentage of sewage in the mixture, gives the total oxygen demand of the sewage or effluent. Presumably the necessary dilution required by the sewage can be calculated from this result, the effect of re-aeration being left as a factor of safety. Unfortunately, however, it has been proven that this demand, as calculated from various dilutions, does not remain constant, but is usually much higher when calculated from high dilutions. The English chemists claim that the demand will remain fairly constant if such dilutions are chosen that from 50–60 per cent of the initial oxygen is consumed, but in America considerable variation has been found in the results, even when confined to these limits (Lederer, 1914). The test is still used to a considerable extent, notwithstanding its inaccuracies, but has not been included in the Standard Methods of the American Public Health Association for 1917.

Two other methods have been given the official sanction of the Committee on Standard Methods — the methylene blue method, to be discussed later, and the sodium nitrate method, which was developed by Dr. Arthur Lederer.

In the sodium nitrate method, an excess of sodium nitrate is added to the undiluted sewage in a closed bottle, and after 10 days' incubation at 37° C. or 20 days' at 20° C., the residual oxygen present as nitrites and nitrates is determined. The amount destroyed by the organic matter and bacteria is the oxygen demand of the sewage. This method has the advantage of being simple, and the technique of applying it is not as difficult as is that of the dilution method. It gives concordant results when used by skilled workers, but it is doubtful whether the same reactions take place with oxygen from nitrates as occur

with dissolved oxygen. There is also difficulty in determining the enormously high quantities of nitrite nitrogen in the residual liquid.

The fact is that no easy, satisfactory biochemical method has yet been developed for determining the dissolved oxygen requirement of sewage. Much progress has been made in this direction within the last decade, however, and the new sewage biochemistry is being developed on a more accurate basis.

The Methylene Blue Test. To obtain results by the oxygen consumed, the oxygen dissolved, or the sodium nitrate method requires not only a well-developed laboratory, but also chemical training, and on this account these tests are not well adapted for the daily testing of effluents at many sewage disposal plants. Methods which require only simple apparatus and no special training have been proposed from time to time, the best being the test, originally devised by Sp.tta (1901) and improved by Spitta and Weldert (1906).

This test depends upon the fact that methylene blue, the zinc salt of tetramethylthionine chloride ($C_{16}H_{18}N_3SCl$), or methylene green, the zinc salt of mononitromethylene blue, are usually broken down, in the absence of oxygen, into colorless derivatives by various substances formed during the decomposition of the organic matter contained in sewage, and especially by hydrogen sulphide. The method consists in adding a small amount of the methylene blue or methylene green, dissolved in water, to the effluent, in a tightly stoppered bottle, and noting the number of days required for the color to be discharged. The time required for decolorization depends of course upon the temperature, being about twice as long at 70° F. as at 98° F.

According to the 1917 Report on Standard Methods the methylene blue (preferably the double zinc salt or commercial variety) should be made up in a 0.05 per cent aqueous solution and exactly 0.4 c.c. of this solution should be added to 150 c.c. of the sample in a glass-stoppered bottle. Incubation is usually for ten days at 20 degrees although 4 days' stability may be considered as sufficient in ordinary routine control. Approximate results may be obtained by incubating at 37° C. for 5 days.

Table CXXXVI gives the relative stability numbers corresponding to observed decolorization after various periods at 20° C. — the theoretical relation as worked out by Prof. E. B. Phelps being $S = 100\ (1 - 0.794\ t_{20})$.

TABLE CXXXVI

RELATIVE STABILITY NUMBERS

Time required for decolorization in days at 20° C.	Relative stability.	Time required for decolorization in days at 20° C.	Relative stability.
0.5	11	8	84
1	21	9	87
1.5	30	10	90
2	37	11	92
2.5	44	12	94
3	50	13	95
4	60	14	96
5	68	16	97
6	75	18	98
7	80	20	99

The oxygen demand of an effluent may be determined from the relative stability and the available oxygen present as dissolved oxygen, nitrites and nitrates by dividing the latter value by the former expressed as a decimal. For sewages and clarified effluents dilutions are made with aerated distilled water of known oxygen content, such that the relative stability of the mixture will be between 50 and 75; and the oxygen demand is then calculated by the formula

$$OD = \frac{O\,(1 - p)}{Rp}$$

where O is the initial dissolved oxygen of the diluting water; p the proportion of sewage; R the relative stability of the mixture; and OD the oxygen demand in parts per million.

The methylene blue test, like the oxygen consumed and the organic sulphur method, has been criticised on the ground that it sometimes indicates putrescibility when the effluent is really stable. This is probably due to the fact that in certain cases the amount of decomposable organic matter remaining in the effluent after the oxygen has all been used up, even though sufficient to reduce methylene blue, is so small in amount that during its decomposition, even under anaerobic conditions, no noticeable odor is developed.

Another criticism is sometimes made to the effect that substances like iron sulphide and hydrogen sulphide bring about a quicker discharge of color than putrescible organic compounds.

There is very little force in this criticism, for hydrogen sulphide, according to most authorities, is one of the products always developed in putrescible effluents; and if effluents contain iron sulphide and hydrogen sulphide they are certainly in an unstable condition.

Bacteriological Methods. In making bacteriological examinations of sewage and sewage effluents the random variations to be expected are so great that refinements necessary in sanitary water analysis are not essential. It is common in ordinary practice to use agar instead of gelatin for plating and for the purpose of measuring the purification effected by treatment devices incubation at 37° C. is as satisfactory as the 20 degrees incubation used in water work. It will yield some additional information with slight trouble if litmus lactose agar be used instead of plain agar so that the number of acid forming organisms can be determined along with the total count.

For the estimation of the intestinal bacteria the presumptive test for gas formation in lactose broth will generally suffice in routine sewage work.

American Public Health Association Tests. The Committee on Sewage Works Operation of the Sanitary Engineering Section of the American Public Health Association, in its 1914 Progress Report (C. S. W. O., 1915) recommended the following tests as of special importance; an incubation test to measure the avidity of the sample of sewage or effluent for dissolved oxygen or its equivalent (called by the Committee Test X); a test for measuring the amount, condition and physical characteristics of the suspended matter present (called by the Committee Test Y); nitrogen as nitrates; dissolved oxygen; relative stability; specific gravity; moisture content, volatile fraction and alkalinity or acidity of sludge; and presumptive test for *B. coli* (when bacterial purity is in question). These tests are to be made as a regular routine and occasional determinations should also be made of the organic and ammonia nitrogen and chlorine in the crude sewage.

The table below is given by the Committee as indicating the conditions under which these various tests should be applied.

Tests for determining the composition of crude sewage for comparison. Occasional composite samples to be analyzed for nitrogen as organic and as ammonia nitrogen. Test for suspended matter called Y in report and chlorine at same time. Samples to be examined for avidity for oxygen, test called X in report.

For sewage treatment works primarily designed to prevent nuisance.

Type of works.	Tests for operating control and determination of efficiency.				Tests to determine sufficiency of treatment.
	Influent and effluent of preliminary process.	Sludge drawn from: One story tanks. / Two story tanks.	Effluent of oxidation process.	Final effluent as discharged into the receiving body of water.	Receiving body of water.
Works consisting of preliminary processes only such as fine screens or sedimentation.	(1) Test for the avidity for oxygen called *X* in text.	(1) Specific gravity. (2) Percentage moisture in wet sludge. (3) Percentage of dry residue that is volatile. (4) Reaction of wet sludge.		(1) Test for avidity for oxygen called *X* in text. (2) Test for suspended matter called *Y* in text.	(1) Dissolved oxygen.
Works consisting of preliminary and oxidation processes such as contact beds or percolating filters.	(2) Test for suspended matter called *Y* in text.		(1) Nitrogen as nitrates. (2) Test for the avidity for oxygen called *X* in text. (3) Dissolved oxygen. (4) Relative stability.*	(1) Test for avidity for oxygen called *X* in text. (2) Nitrogen as nitrates. (3) Dissolved oxygen. (4) Test for suspended matter called *Y* in text (5) Relative stability.*	(2) Test for avidity for oxygen called *X* in text.

For sewage treatment works primarily designed to protect sources of water supply.

Works consisting of preliminary processes and disinfection.	(1) Test for the avidity for oxygen called *X* in text.	(1) Specific gravity. (2) Percentage moisture in wet sludge. (3) Percentage of dry residue that is volatile. (4) Reaction of wet sludge.		(1) Presumptive test for *B. coli*. (2) Test for avidity for oxygen called *X* in text. (3) Test for suspended matter called *Y* in text.	(1) Presumptive test for *B. coli*. (2) Dissolved oxygen. (3) Test for avidity for oxygen called *X* in text.
Works consisting of preliminary and oxidation processes and disinfection.	(2) Test for suspended matter called *Y* in text.		(1) Nitrogen as nitrates. (2) Test for avidity for oxygen called *X* in text. (3) Dissolved oxygen. (4) Relative stability.*	(1) Presumptive test for *B. coli*. (2) Test for avidity for oxygen called *X* in text. (3) Nitrogen as nitrates. (4) Test for suspended matter called *Y* in text. (5) Relative stability.* (6) Dissolved oxygen.	

* In small plants not provided with a laboratory relative stability may take first place.

Standards of Purity. Various standards of purity have from time to time been formulated in England, and the different views held as to when an effluent should be deemed polluting and inadmissible into a stream are shown by the provisional standards of purity of certain well-known commissions. The Mersey and Irwell Joint Board required that the effluent should not contain, per million parts, more than 1.4 parts albuminoid nitrogen, 14.3 parts oxygen absorbed in 4 hours at 60° F., 3.6 parts oxygen absorbed in 3 minutes at 60° F., and 43 parts solids in suspension, and that it shall be nonputrescible by incubator test for 3 to 5 days at 75° F. The Ribble Joint Committee considered an effluent as unsatisfactory if the albuminoid nitrogen ranged between 1.5 and 2 parts per million, and bad if the albuminoid nitrogen were over 2 parts, and required that the suspended matter should not exceed 30 parts per million. The Thames Conservancy Board deemed an effluent containing 30 parts of organic carbon and 11 parts of organic nitrogen per million unsatisfactory.

The latest and most authoritative standard of this kind is the one formulated by the Royal Commission on Sewage Disposal in its Eighth Report (R. S. C., 1912). It provides as a general standard that a sewage effluent must not contain as discharged more than 30 parts per million of suspended matter and must not take up more than 30 parts per million of dissolved oxygen in 5 days at 18.3° C. If the volume of diluting water available is 150–300 times the volume of the effluent the dissolved oxygen test may be omitted and 60 parts of suspended solids would be permissible; if the volume of diluting water is 300–500 times the volume of the effluent the limit of suspended solids may be raised to 150 parts; while if the diluting volume is 500 times that of the effluent all standards might be dispensed with provided necessary screens and detritus tanks were provided.

The desirability of such standards of purity, to which all sewage effluents should be made to conform, has always been somewhat questionable, the inherent difficulty being that no account is taken in such standards of varying local conditions and the uses to be made of the water into which the effluent is discharged. Furthermore, the effluent of a trickling filter containing an amount of organic matter considerably in excess of the above

standards may often be stable on account of the simultaneous presence of a large amount of dissolved oxygen. Consequently such standards have not been generally recognized in America. In the United States the usual requirement is that the effluent must be nonputrescible, or must contain only that amount of organic matter which, when the effluent is emptied into a stream, can be oxidized by the oxygen contained in the water at the time of the minimum flow; and that it must be sufficiently freed from suspended solids to avoid the accumulation of sludge banks or the creation of surface conditions offensive to sight and smell. In certain cases, as when the effluent is emptied into a stream in the near neighborhood of a point at which water is taken for domestic use, or is emptied into salt water in the neighborhood of shellfish layings, the effluent must also be freed from the great majority of the sewage bacteria.

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